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DELINEATION OF FLOODPLAINS FOR THE UNNAMED WASH IN MOAB CANYON, ARCHES NATIONAL PARK, UTAH

William B. Reed

Technical Report NPS/NRWRD/NRTR-90/03

WATER RESOURCES DIVISION

National Park Service • Department of Interior
Fort Collins • Denver • Washington

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Water Resources Division • 301 S. Howes Street • Fort Collins, Colorado 80521

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IN MOAB CANYON, ARCHES NATIONAL PARK, UTAH

ABSTRACT

A hydraulic analysis of the unnamed wash in Moab Canyon at Arches National Park, Utah, was made using the U.S. Army Corps of Engineers' HEC-2 (Water Surface Profiles) computer program to calculate peak water-surface elevations for the 100-year, 500-year, and probable maximum flood. The section of the unnamed wash studied was a 3,920-foot reach in the vicinity of the park's main entrance. Of the nine major buildings in this area, six were found to be subject to flooding. Alternatives for managing described flood hazards are presented.

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ACKNOWLEDGEMENTS

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INTRODUCTION

On August 27, 1985, the Park Planning Division, Rocky Mountain Region, National Park Service, requested assistance from the Water Resources Division in quantifying flood hazards for Arches National Park. The analysis was required as part of the general management planning process for the park. Specifically, Region and park staff were concerned that both existing and planned facilities might be subject to hazardous flooding.

At the time of this request, existing structures included the park visitor center, the park maintenance facility, the Natural History Association office, and six park residential buildings. Planned facilities included an expanded visitor contact area with a larger visitor center/park headquarters complex.

The purpose of this study is to delineate for the unnamed wash in Moab Canyon the floodplains for the 100-year, 500-year, and probable maximum flood (PMF), *and* to assess the flood risk to existing and planned facilities. Additionally, alternatives for managing the described flood hazards are provided.

DESCRIPTION OF STUDY AREA

GEOGRAPHIC SETTING

Arches National Park consists of 7,379 acres (13.9 square miles) in southeastern Utah. Within the park are natural stone arches, windows, pinnacles, and other unique natural features indigenous to the area. The studied wash is in Moab Canyon about 5 miles northwest of Moab, Utah (Figures 1 and 2). Within the canyon, U.S. Highway 191 parallels this wash, sometimes referred to as Bloody Mary Wash. This wash is ephemeral and tributary to the Colorado River.

The main egress to the park crosses the unnamed wash near the park visitor center. The bridge is a culvert reinforced stone arch with a width of 20 feet, a length of 66 feet, and a height of 11 feet. The top of the roadway over the arch is higher than a dip in the road upstream.

WATERSHED

The watershed is small and characterized by bare rock, thin soils, and sparse desert vegetation. The net result of these watershed characteristics is rapid runoff in response to heavy rains. This is typical of deserts of the southwestern United States; a region that is especially subject to flash floods. Although it is most likely that a life-threatening flood (a large magnitude flash flood) will occur during the summer months as a result of intensive rainfall, such a flood can occur at anytime during the year--day or night--spring, summer, fall, or winter.

The drainage area for the wash above a point southeast (downstream) of park facilities (total basin) is 4.88 square miles (Figure 3). This drainage area has a mean watershed elevation of 4,524.6 feet Mean Sea Level (MSL). The drainage area for the wash above a point west (upstream) of park facilities (upper basin) is 3.86 square miles. This is the shaded area on Figure 3. The mean watershed elevation of this basin segment is 4,525 feet.

STUDY REACH

The channel reach selected for analysis is 3,920 feet in length, measured along the thalweg. The study reach begins 450 feet upstream of the overflow parking lot, and ends 1,200 feet downstream from the maintenance yard (Figure 4).

The upper section of this reach trends from west to east--upstream of the entrance road bridge that crosses the wash downstream of the park visitor center. This section begins below a waterfall near its plunge pool. Immediately downstream of the plunge pool, the channel widens, but is contained by U.S. Highway 191 on the south (right bank) and the park entrance road on the north (left bank). Here the channel has been straightened and stabilized.

The lower section of the study reach trends from northwest to southeast--downstream of the entrance road bridge. Immediately downstream of the bridge, the channel widens to the confluence with the main tributary along this reach. From this point to the end of the reach, the main channel is only slightly entrenched, and downstream of the park maintenance facilities, the active floodplain occupies a wide expanse between the highway berm and the sandstone escarpment on the north side of the canyon.

MAIN TRIBUTARY

The main tributary drains the steep sandstone area north and east of the visitor center complex. In the vicinity of the park visitor center, a ditch has been built, parallel to the escarpment, to convey the tributary and water that historically flowed to the main channel via several small channels. The ditch conveys these waters to the largest of the former small channels, east of the visitor center. From this point, the tributary trends southeast to the confluence with the main channel below the entrance road bridge. This tributary system is particularly important to the floodplain evaluation due to its proximity to several park residences at the base of the sandstone escarpment (Figure 4).

BUILDINGS

All buildings within the study area are to the north of the study reach (Figure 4). From west to east, the buildings and other park facilities are: (1) overflow parking lot (dirt), (2) main parking lot (paved), (3) park visitor center, (4) a building circa 1940s (park library, Natural History Association office, and storage for office and museum), (5) park residence #1, (6) park dormitory (multifamily residence), (7) park residence #2 (roughly due north of the bridge), (8) park residence #3, (9) park residence #4, (10) park residence #5, and (11) the park maintenance facility.

STUDY METHODS

FLOOD HYDROLOGY

Peak discharge values calculated for the 100-year flood, 500-year flood, and PMF and concurrent flow values for the main tributary are listed in Table 1. These values are in cubic feet per second (cfs).

Table 1. PEAK DISCHARGE ESTIMATES FOR THE UNNAMED WASH AND CONCURRENT FLOW VALUES FOR THE MAIN TRIBUTARY

Type of Event	Main Channel (upper basin)	Main Tributary	Main Channel (total basin)
100-year	5,100 cfs	500 cfs	5,600 cfs
500-year	8,600 cfs	800 cfs	9,400 cfs
PMF	19,200 cfs	1,900 cfs	21,100 cfs

The peak discharges for the 100- and 500-year floods were calculated using the Low Plateaus Region methods developed by Thomas and Lindskov (1983). For the main channel upstream of the entrance road bridge, basin parameters for the drainage area upstream of the study reach (upper basin) were used (shaded area on Figure 3). For the study reach downstream of the bridge, basin parameters for the drainage area upstream of the lowermost cross section (total basin) were used.

To obtain the concurrent flow values for the main tributary associated with the 100- and 500-year floods, the peak discharge values for the drainage area upstream of the study reach were subtracted from the values for the drainage area upstream of the lowermost cross section.

To determine the PMF peak discharge value for the upper basin, the local-storm probable maximum precipitation (PMP) was calculated and a storm hydrograph developed. The local-storm PMP (an estimate of the upper limit of rainfall resulting from summer or early fall local storms) with a duration of 1 hour was calculated using the method developed by Hansen, et al. (1977). The areal-reduced PMP was calculated to be 7.6 inches with the following 15-minute time increments (listed in sequence of occurrence): 6.1 inches, 0.7 inches, 0.4 inches, and 0.4 inches.

A storm hydrograph (Figure 5) for the main channel upstream of the entrance road bridge was then derived using the Soil Conservation Service's unit hydrograph method (McCuen, 1982). For this method a drainage area of 3.86 square miles, a time of concentration of 0.86 hours, and a curve number of 95 were used. The time of concentration for the upper basin was determined by using an average channel slope of 0.03 feet/feet and a hydraulic length of 19,008 feet. The curve number is a value slightly lower than the curve number for bare rock (Van Haveren, 1986) which is the predominate land surface within the watershed.

To obtain the concurrent flow value for the main tributary associated with the PMF, the following equation was used:

$$a = b/c \times d$$

where

a = concurrent flow for main tributary associated with PMF;

b = PMF peak discharge for upper basin;

c = 100-year peak discharge for upper basin; and

d = concurrent flow for main tributary associated with 100-year flood.

The (total basin) PMF peak discharge value for the main channel downstream of the bridge was then calculated by adding the concurrent flow to the PMF peak discharge value determined for the upper basin.

HYDRAULIC MODELING

A hydraulic analysis of the unnamed wash and its main tributary was made using the U.S. Army Corps of Engineers' HEC-2 (Water Surface Profiles) computer program (U.S. Army Corps of Engineers, 1982) to calculate peak water-surface elevations for the 100-year, 500-year, and PMF.

Twenty-five cross sections were surveyed to provide hydraulic geometry for the model. The cross sections were located at changes in channel slope, roughness, or shape, as well as at tributary confluences and upstream and downstream from the entrance road bridge (see Appendix). Cross sections were numbered sequentially from the lowest (1) to the highest (25). Cross section 3 was discarded after the survey data had been reduced because it crossed another cross section on the right overbank. This occurred because in the HEC-2 program a transect can be dog-legged to ensure that each segment (channel and left and right overbank) is perpendicular to the direction of flow. However, the data for cross section 3 was reviewed to ensure that no significant hydraulic information was lost when the cross section was discarded.

The length between cross sections was measured along the channel thalweg and from the midpoint of the left and right overbank transect. Thalweg lengths were measured in the field using a steel tape which was curved to follow the path of the deepest part of the channel. Overbank distances were determined in the office using survey information. The dimensions of the bridge opening were measured by steel tape. The bridge roadway elevations were determined by a survey.

Roughness coefficients (Manning's "n" values) were estimated in the field individually for the main channel and left and right overbank at each cross section. These values were selected based on the limited experience of the field personnel, for example, the flood study done by Reed (1986), and by using the following two references: Barnes (1967), and Benson and Dalrymple (1967).

Subcritical flow was assumed for all discharges. This assumption was made because: (1) during floods the reach upstream of the bridge is an area of backwater dominated by subcritical flow and (2) assuming subcritical flow for the reach downstream of the bridge resulted in a conservative estimate and simplified the modeling procedure.

For subcritical flow conditions it is necessary to initiate HEC-2 calculations beginning at the most downstream cross section. For this study the critical flow depth was used. Upstream of the bridge, the discharge in the main channel was set equal to the upper basin discharge. Downstream of the bridge, the discharge in the main channel was set equal to the total basin discharge.

Figure 6 is a visual representation of the conceptual model for routing flood flows in the vicinity of the entrance road bridge. The bridge was modeled using the special bridge option. This modeling option allows weir flow over the road, as well as pressure flow through the bridge opening if necessary. However, some discharge is lost during high flows over the road upstream of the bridge. Flow lost here enters the tributary channel and reenters the main channel downstream of the bridge. Therefore, this roadway overtopping and resulting temporary loss of flow in the main channel was simulated using the split-flow option. The discharge through the bridge opening (located upstream of the tributary) was equal to the upper basin discharge minus the discharge over the roadway into the main tributary.

The main tributary in the vicinity of the bridge was modeled separately. The flow in the tributary was set equal to the concurrent discharge for the tributary plus the flow from the main channel as determined by the split-flow option during the main channel simulation. The starting water-surface elevation for this simulation was set equal to the water-surface elevation at cross section 14 (where the main tributary joins the main channel at high flows). This value was obtained from the main channel simulation.

RESULTS

FLOOD HYDRAULICS

Figure 7 shows the areas that will be inundated by the 100-year flood, 500-year flood, and PMF. This figure was prepared from the results of the hydraulic analysis.

The water-surface profiles for the 100-year flood, 500-year flood, and PMF are shown on Figures 8, 9, and 10 (see Appendix). The streambed profile shown on these figures is based on the lowest point in the streambed at each cross section. The cross sections and water-surface elevations are shown on Figures 11 to 34 (see Appendix). Cross section 3, which was not used in the hydraulic analysis, is not shown. From Figures 8 to 34, the flood depths at any point along the study reach may be determined.

The average depth of the PMF is 10.77 feet. The depth ranges from a low of 6.43 feet at cross section 4 to a high of 15.29 feet at cross section 15. The average depth of the 500-year flood is 7.87 feet. The depth ranges from a low of 4.44 feet at cross section 4 to a high of 12.72 feet at cross section 15. The average depth of the 100-year flood is 6.45 feet. The depth ranges from a low of 3.59 feet at cross section 4 to a high of 11.05 feet at cross section 15.

Cross section 4 has the smallest flood depth for the calculated floods because it is near the end of the study reach where cross sections are broad and vegetation is especially sparse. Cross section 15 has the greatest flood depth for the calculated floods because it is immediately upstream from the bridge where the backwater effect is greatest.

The mean velocities of the 100-year flood, 500-year flood, and PMF for the main channel and left overbank at cross sections 1 through 25 are shown in Table 2. The average main channel mean velocity for the 100-year flood is 10.29 feet/second. The average main channel mean velocity for the 500-year flood is 10.43 feet/second. The average main channel mean velocity for the PMF is 11.46 feet/second. The left overbank velocities for the various floods are discussed in the following flood hazard assessment section.

FLOOD HAZARD ASSESSMENT

Flood depths and corresponding left-overbank mean velocities at the 11 locations (shown on Figure 4) are presented in Table 3. The left-overbank mean velocities were either selected from the main channel *or* tributary computer simulation for the nearest cross section to the subject park facility. Although these values are mean values for a subunit of the cross section, they are provided as an *estimate* of velocities at the facilities.

During a 100-year flood, the dormitory (multifamily residence at cross section 16) and residence 2 (residence at cross section 15) are within the floodplain. During a 500-year flood, three additional locations are within the floodplain: a portion of the visitor center overflow parking lot (dirt), residence 1 (residence at cross section 17), and the maintenance yard complex. During a PMF, only residences 3, 4, and 5 of the 11 locations would *not* be within the floodplain.

Because subcritical flow was assumed for the entire study reach, including the sub-reaches where *supercritical* flow could occur, the velocities in Table 3 may be unrealistically low *and* not indicative of the floods' erosional capabilities. However, this assumption does not affect the overall conclusion of this study; of the nine major buildings in the study area, only three are not subject to flooding.

If a location is within a floodplain, then risk to life and property can be considered *high* at those sites during floods of equal or greater magnitude. The exceptions to this observation are the visitor center overflow parking lot and the maintenance yard complex. Although these locations are within the 500-year floodplain, risk would be low at these locations during a 500-year event because of low flood velocities and shallow flood depths. However, evaluation of parking lots should consider that nationwide flood fatality statistics indicate that approximately half of all people who die in floods do so in their vehicle or while trying to flee from a vehicle (Mooney, 1983).

It should be noted that the park entrance road from U.S. Highway 191 crosses the study reach over a bridge that might fail during major floods, i.e., floods such as the 100- and 500-year floods. Additionally, floodwaters of 100- and 500-year floods would flow over the road upon backing up at the bridge. Therefore, the road from cross section 17 to the highway would be hazardous during the 100-year flood. The road from cross

Table 2. FLOOD MEAN VELOCITIES

Cross Section	100-Year Main Channel (feet/second)	100-Year Left Overbank (feet/second)	500-Year Main Channel (feet/second)	500-Year Left Overbank (feet/second)	Probable Maximum Flood Main Channel (feet/second)	Probable Maximum Flood Left Overbank (feet/second)
1	8.46	4.20	9.84	5.62	12.50	7.76
2	7.99	4.80	10.05	6.55	12.71	8.84
4	8.33	5.48	9.74	7.47	12.45	10.60
5	6.35	4.38	7.61	5.54	6.08	4.54
6	8.39	6.69	9.95	7.73	13.19	10.48
7	9.56	7.27	11.06	8.88	6.42	6.31
8	9.90	7.47	11.43	9.10	13.30	12.42
9	8.49	5.39	9.98	6.22	11.96	8.41
10	11.76	6.58	12.71	6.49	15.96	7.67
11	10.85	6.09	12.86	8.84	10.59	14.33
12	10.40	5.97	12.04	7.27	15.70	10.19
13	11.09	6.15	11.41	6.62	12.28	7.74
14	11.78	5.69	13.86	7.07	17.34	9.22
15	13.78	3.57	12.30	7.20	10.44	12.72
16	2.78	11.42	2.41	13.93	1.72	18.96
17	5.52	0.00 (slack water)	2.68	10.28	1.89	14.95
18	8.91	0.00 (no flow)	12.07	0.00 (slack water)	11.62	7.28
19	9.50	0.00 (no flow)	9.17	0.00 (no flow)	10.28	6.02
20	7.25	0.00 (no flow)	8.57	0.00 (no flow)	8.77	5.81
21	9.49	0.00 (no flow)	11.79	0.00 (no flow)	13.97	5.93
22	9.87	0.00 (no flow)	8.48	0.00 (slack water)	8.01	3.60
23	11.45	0.00 (no flow)	13.17	0.00 (slack water)	7.68	4.87
24	11.17	0.00 (no flow)	13.08	0.00 (no flow)	16.37	7.75
25	11.78	0.00 (no flow)	13.96	0.00 (no flow)	17.58	5.54

Table 2. (cont) AVERAGE MEAN VELOCITY

Cross Section	100-Year Main Channel (feet/second)	100-Year Left Overbank (feet/second)	500-Year Main Channel (feet/second)	500-Year Left Overbank (feet/second)	Probable Maximum Flood Main Channel (feet/second)	Probable Maximum Flood Left Overbank (feet/second)
1 - 15	10.36	5.70	11.06	7.19	12.64	9.37
16 - 25	10.18	—	9.54	—	9.81	8.07
1 - 25	10.29	—	10.43	—	11.46	8.83

Table 3. FLOOD DEPTHS AND LEFT-OVERBANK MEAN VELOCITIES AT MAJOR FACILITIES

Facility/Location	100-Year Flood		500-Year Flood		Probable Maximum Flood	
	Depth (feet)	Velocity (feet/sec)	Depth (feet)	Velocity (feet/sec)	Depth (feet)	Velocity (feet/sec)
1. Dirt parking lot	*	*	less than 1.0	0	4.5	3.5
2. Paved parking lot	*	*	*	*	3.0	6.0
3. Visitor center	*	*	*	*	2.0	6.0
4. 1940 house	*	*	*	*	3.0	8.0
5. Residence #1	*	*	less than 2.0	7.0	5.0	9.0
6. Dormitory	less than 2.0	6.0	3.0	8.0	greater than 5.0	11.5
7. Residence #2	less than 2.0	3.0	3.5	5.0	6.0	9.0
8. Residence #3	*	*	*	*	*	*
9. Residence #4	*	*	*	*	*	*
10. Residence #5	*	*	*	*	*	*
11. Maintenance yard and facility	*	*	less than 0.5	6.5	2.0	8.0

* not within floodplain

section 18 to the highway would be hazardous during the 500-year flood, and the road from upstream of cross section 23 to the highway would be hazardous during the PMF. Thus, use of the bridge should be avoided during *all* major flood events. Also, the service road between the visitor center and the maintenance yard would be hazardous during major flood events.

The flood discharges used in the hydraulic analysis are water discharges only. The study did not consider three impacts: (1) the effect of bulking, i.e., what would happen if sediment and/or debris load significantly contributed to total flood discharges; (2) the possible failure of buildings and hydraulic structures as a source of debris; and (3) what the effect of existing buildings would be upon flood flows. These structures could, however, *increase* flood depths in their vicinities, especially immediately upstream, against the walls of the structure.

CONCLUSIONS AND DISCUSSIONS

The unnamed wash in Moab Canyon is subject to hazardous flood flows that would present an immediate danger to park visitors and employees in the vicinity of the park's main entrance. Of the existing buildings in this area, only three are safe from floods. Because this is a high-hazard area, alternatives for relocating the park entrance road and five park buildings (visitor center, Natural History Association office, residence #1, dormitory, and residence #2) should be evaluated; as well as alternatives for providing structural protection for these facilities.

This study assumed there would be no failure of existing hydraulic structures. However, failure probably would occur, particularly during the 500-year event or the PMF. The difficulty of incorporating potential failures into the model is the great uncertainty about when and where failure would occur. However, three failures that could occur are: failure of the ditch for the main tributary in the vicinity of the visitor center, failure of the existing roadway berm upstream of the bridge, and failure of the bridge.

Additional studies which may be done in the future to more thoroughly examine possible flood control measures include:

- detailed engineering analysis of bridge strength and possible failure during floods;

- more detailed hydrologic analysis of the tributary and an engineering analysis of ditch conveyance;

- evaluation of the potential for debris flows and sediment transport; and

- structural mitigation study that evaluates alternatives to the removal or relocation of facilities.

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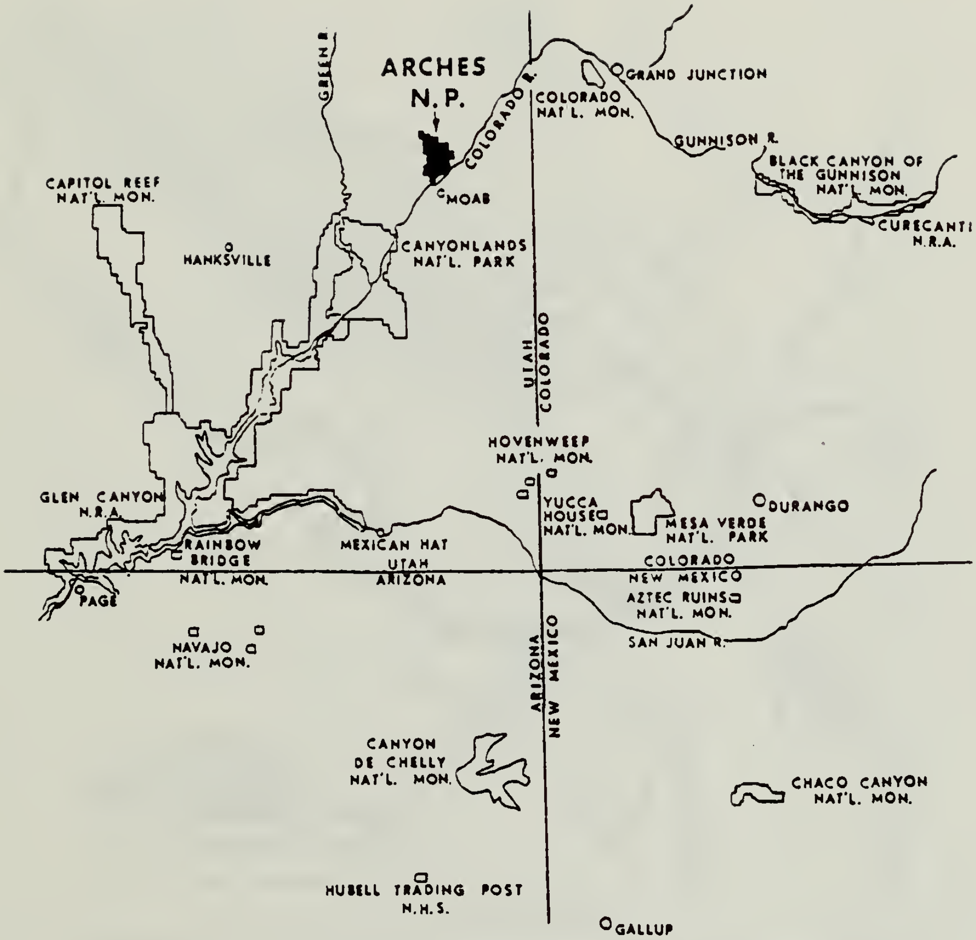
APPENDIX



REGION MAP

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Figure I. Region Map

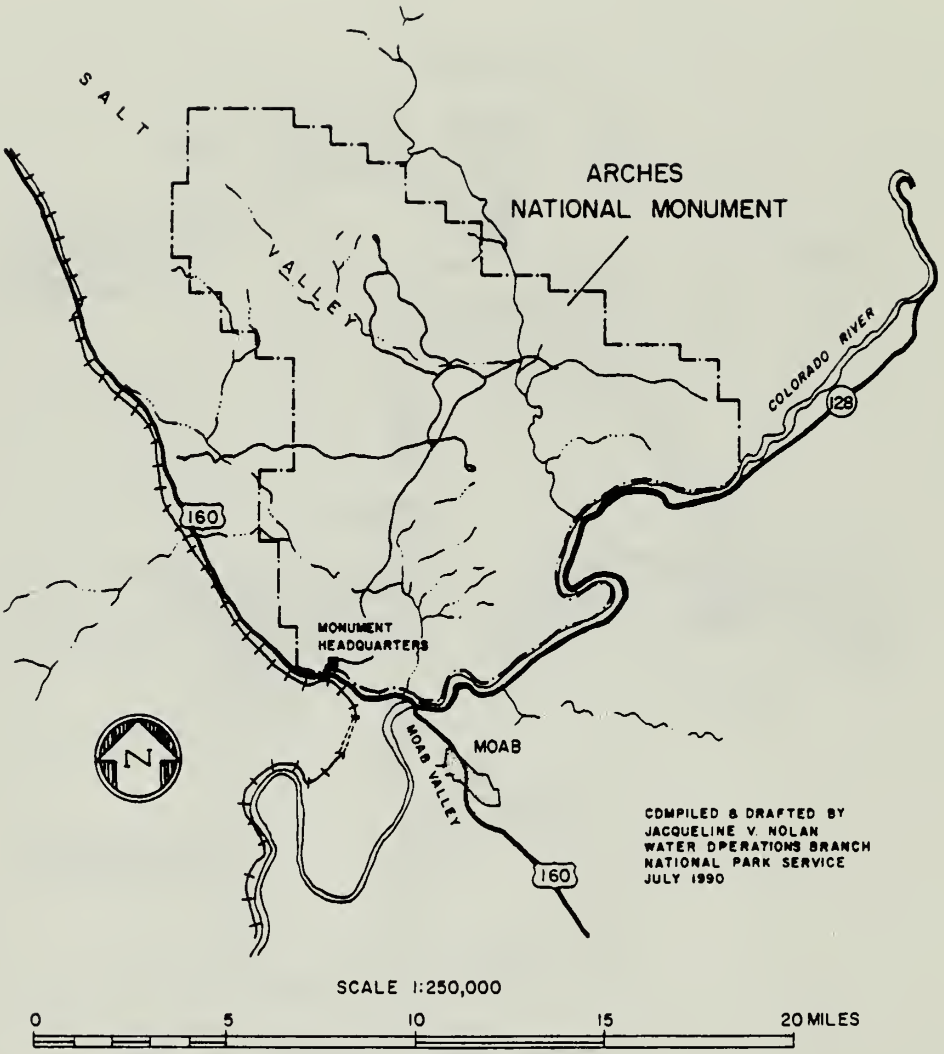
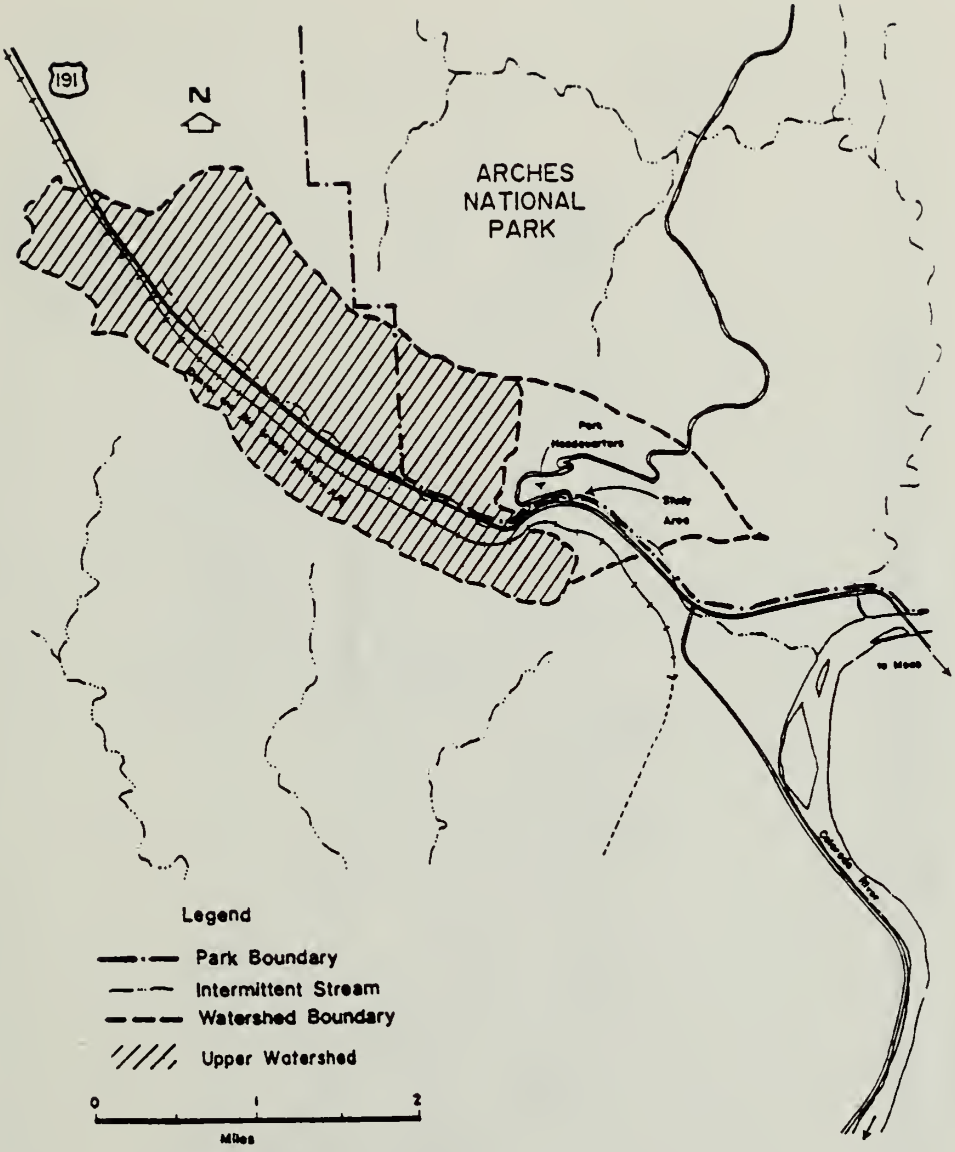


Figure 2. Park Vicinity Map



Contributing Watershed Map

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Figure 3. Contributing Watershed Map

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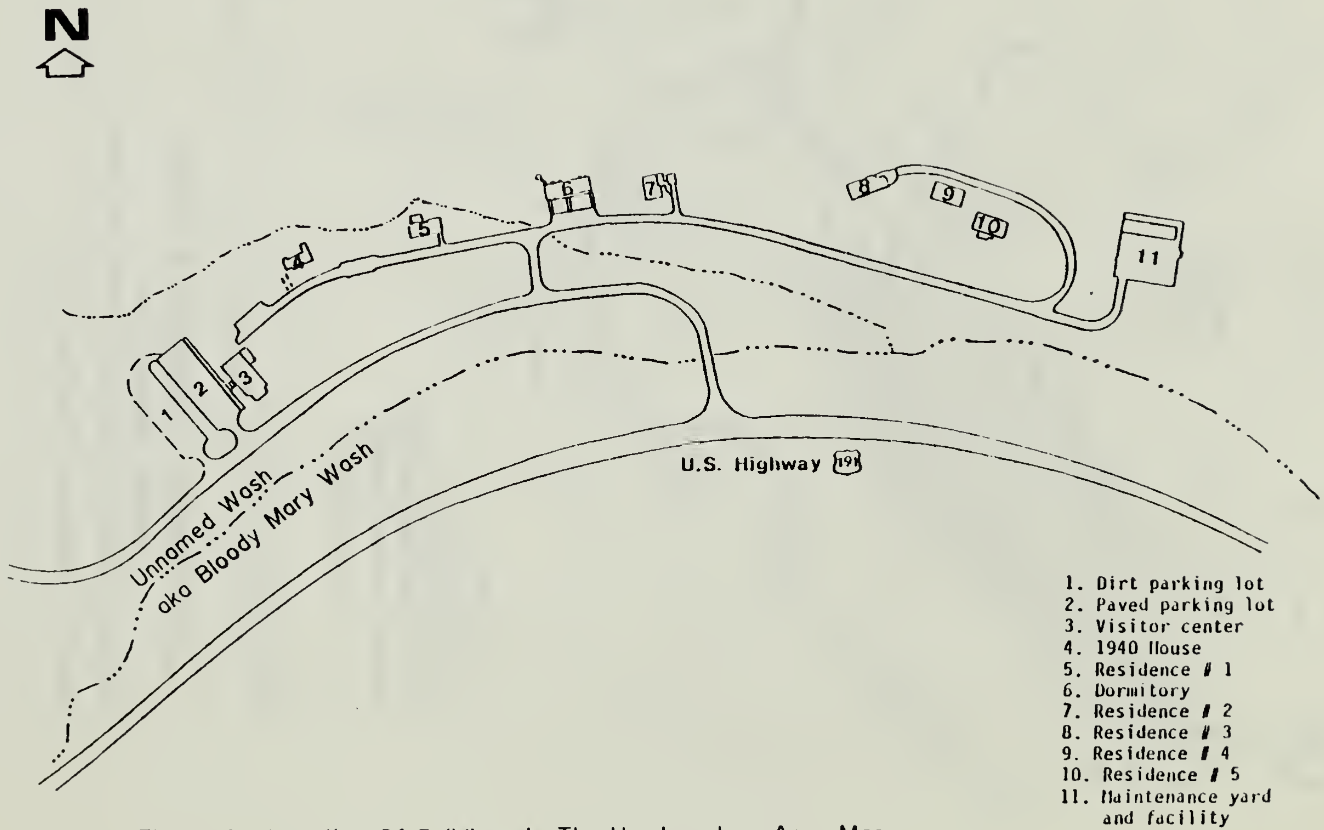

Figure 4. Location Of Buildings In The Headquarters Area Map

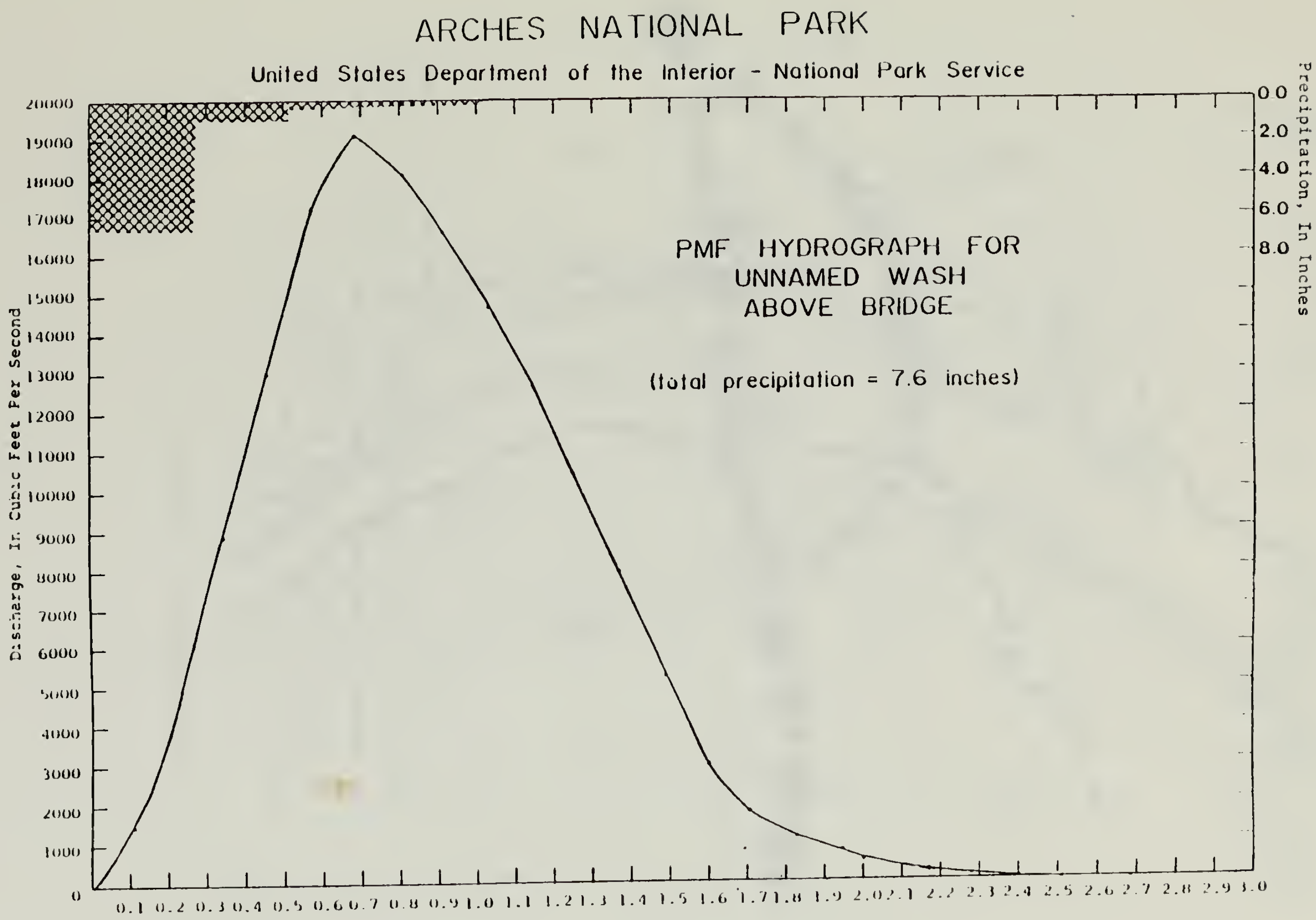

Figure 5. PMF Hydrograph

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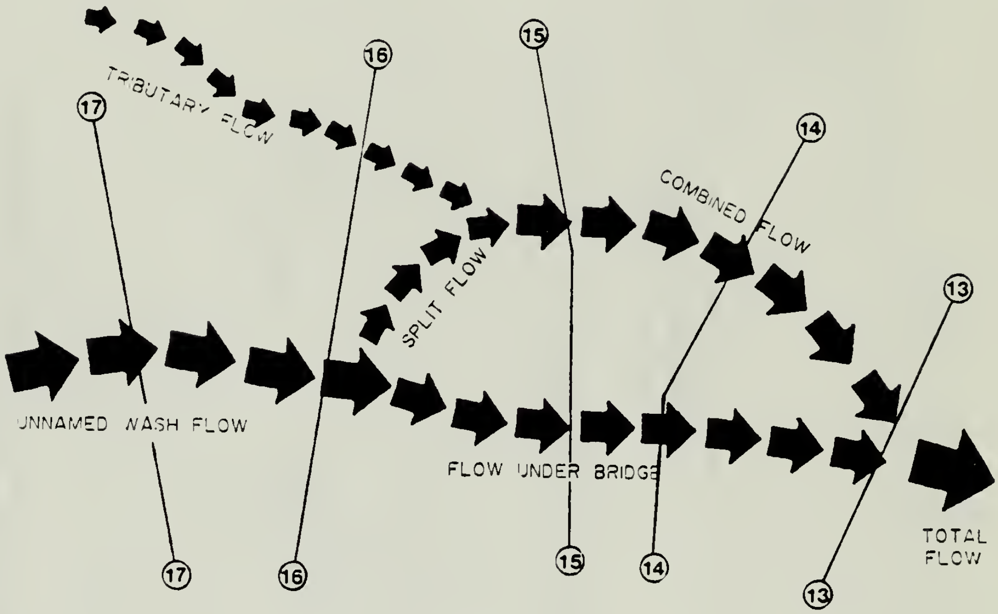

FIGURE 6

Conceptual model for flood flows in the vicinity of the entrance road bridge (plan view) where: 1) Flow under the bridge was determined using HEC-2 special bridge option; 2) Split flow was determined using the HEC-2 split flow option; 3) Combined flow was equal to split flow plus tributary flow; and 4) Total flow was equal to combined flow plus flow under bridge, which is the same as unnamed wash flow plus tributary flow.

Figure 6. Conceptual Model For Flood Flows

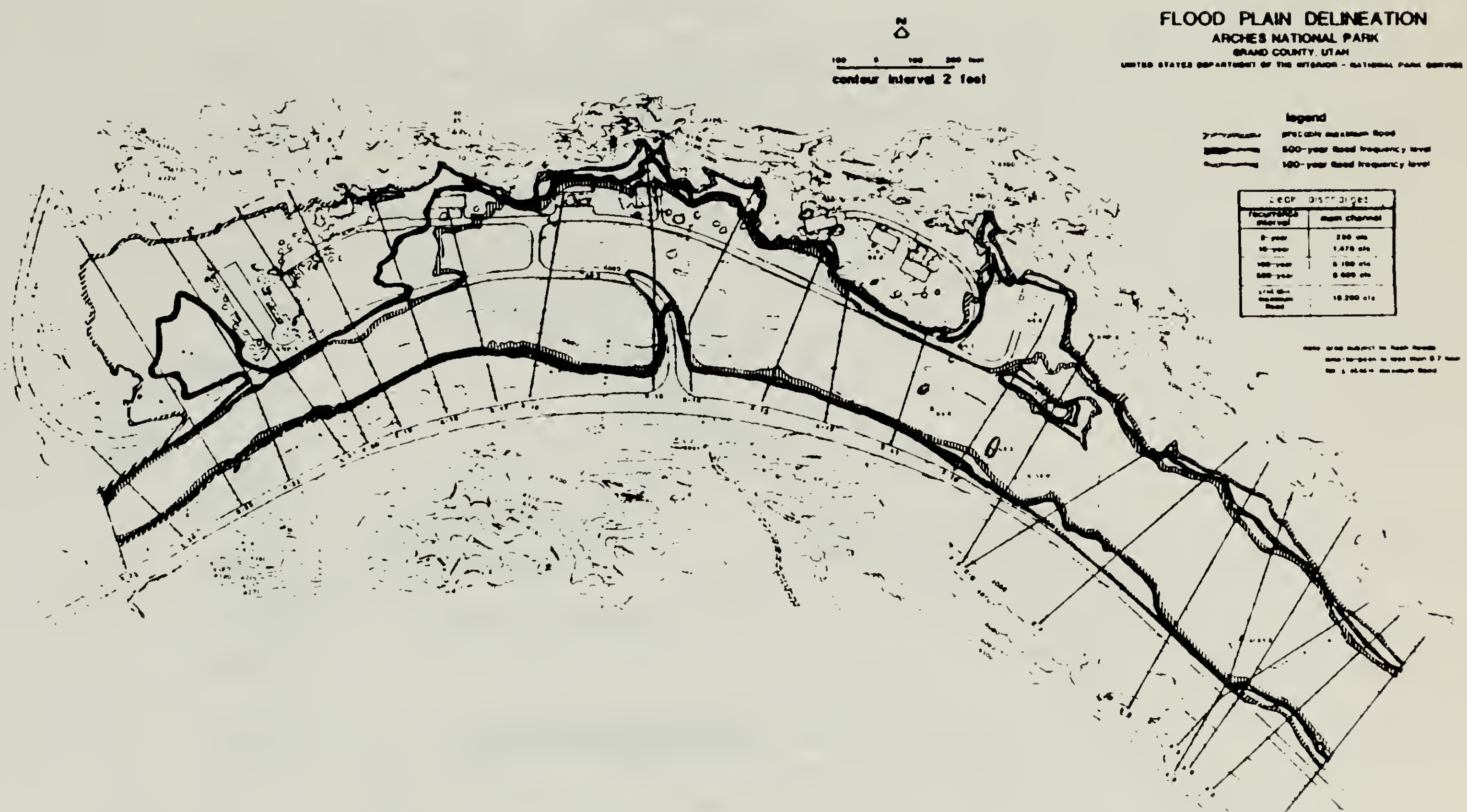

Figure 7. Flood Plain Delineation

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Unnamed Wash

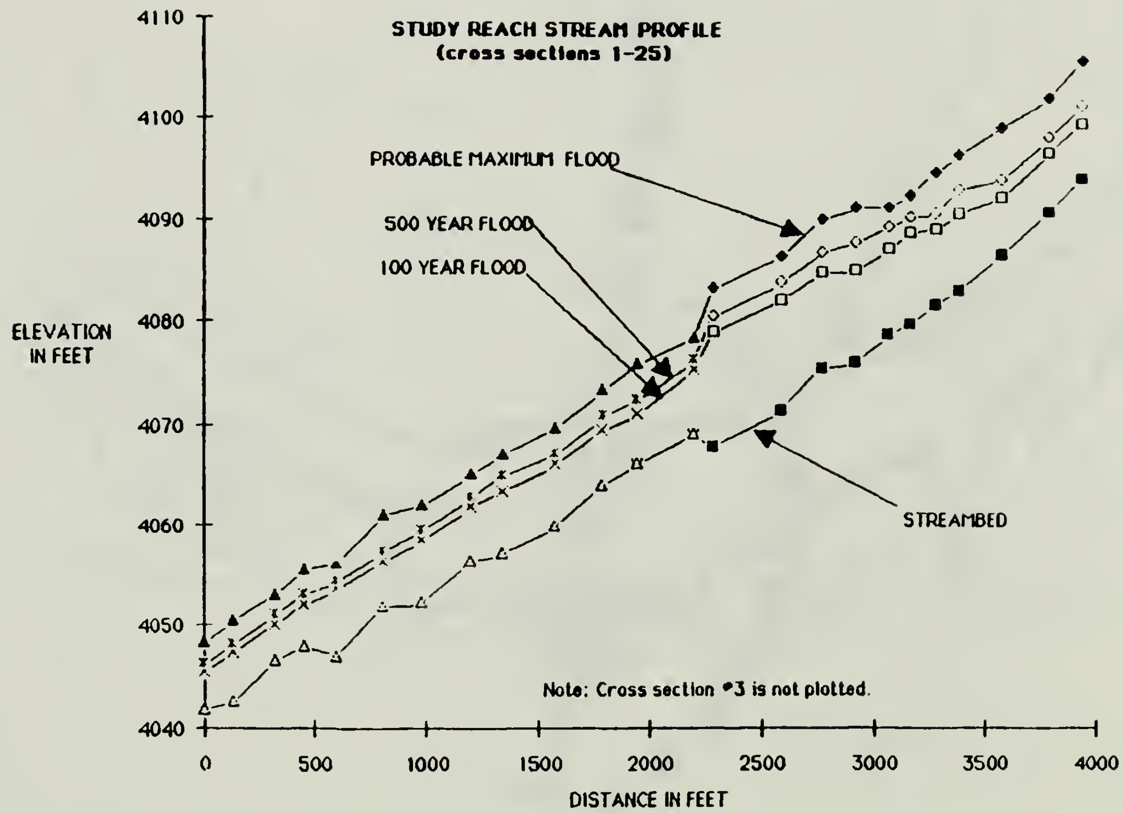

Figure 8. Study Reach Flood and Streambed Profiles

ARCHES NATIONAL PARK

Unnamed Wash

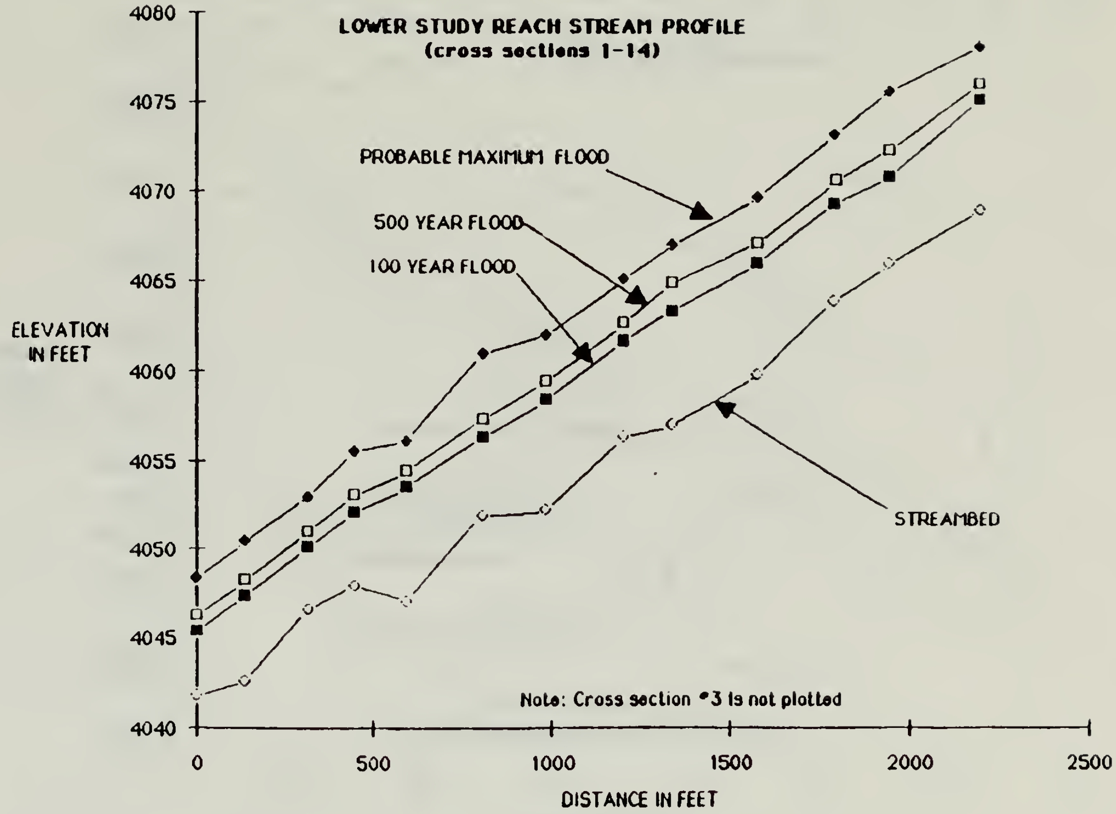

Figure 9. Lower Study Reach Profiles

ARCHES NATIONAL PARK

Unnamed Wash

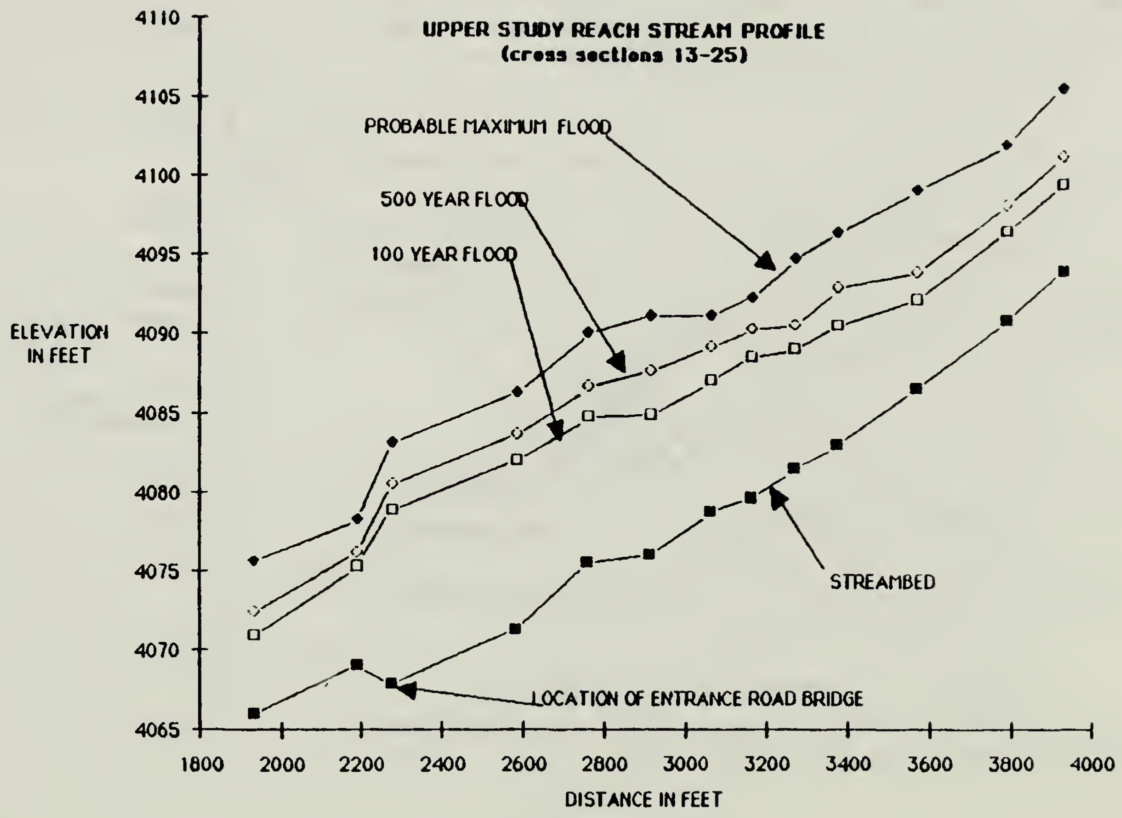

Figure 10. Upper Study Reach Profiles

ARCHES NATIONAL PARK
Unnamed Wash

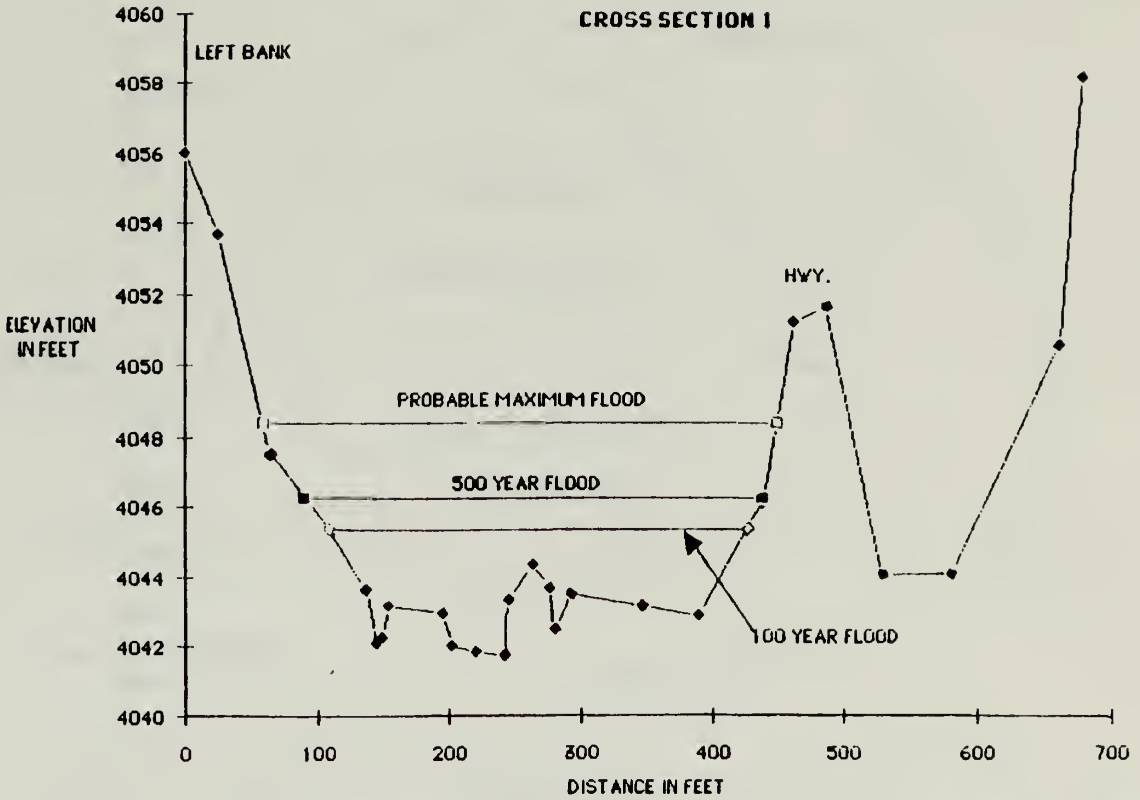

Figure II. Floodwater - Surface Elevations -- Cross Section I

ARCHES NATIONAL PARK

Unnamed Wash

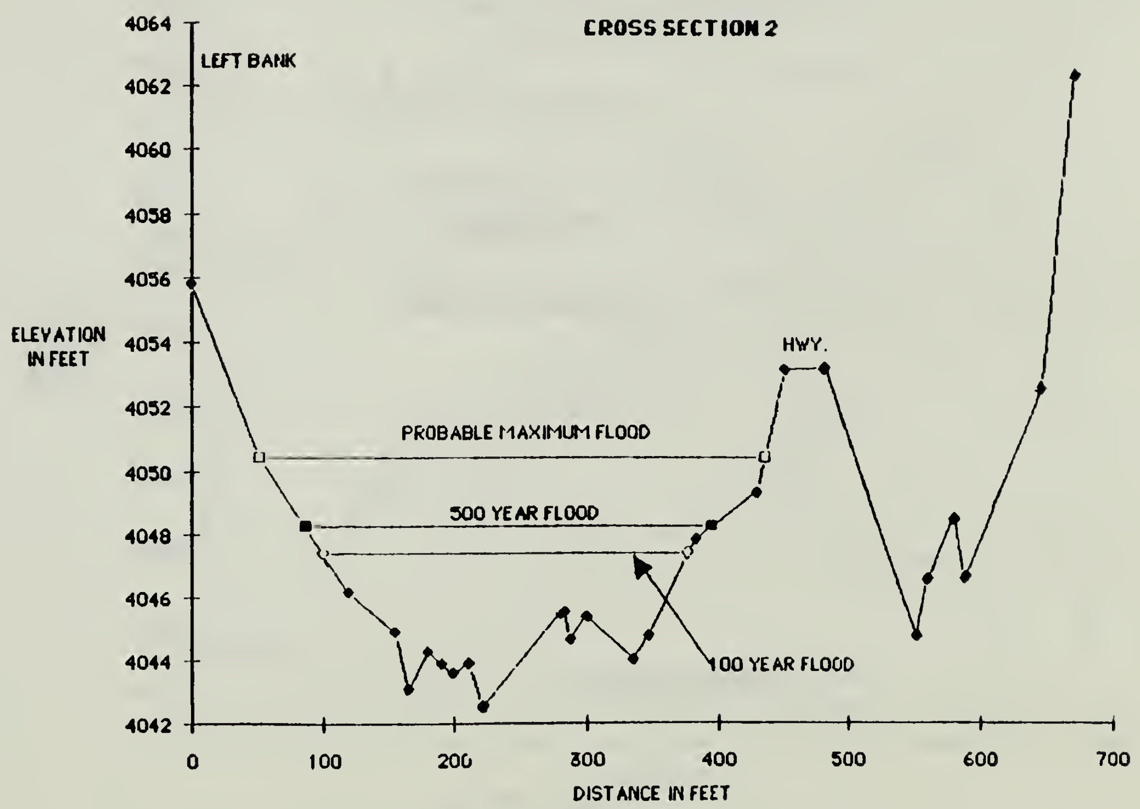

Figure 12. Floodwater - Surface Elevations -- Cross Section 2

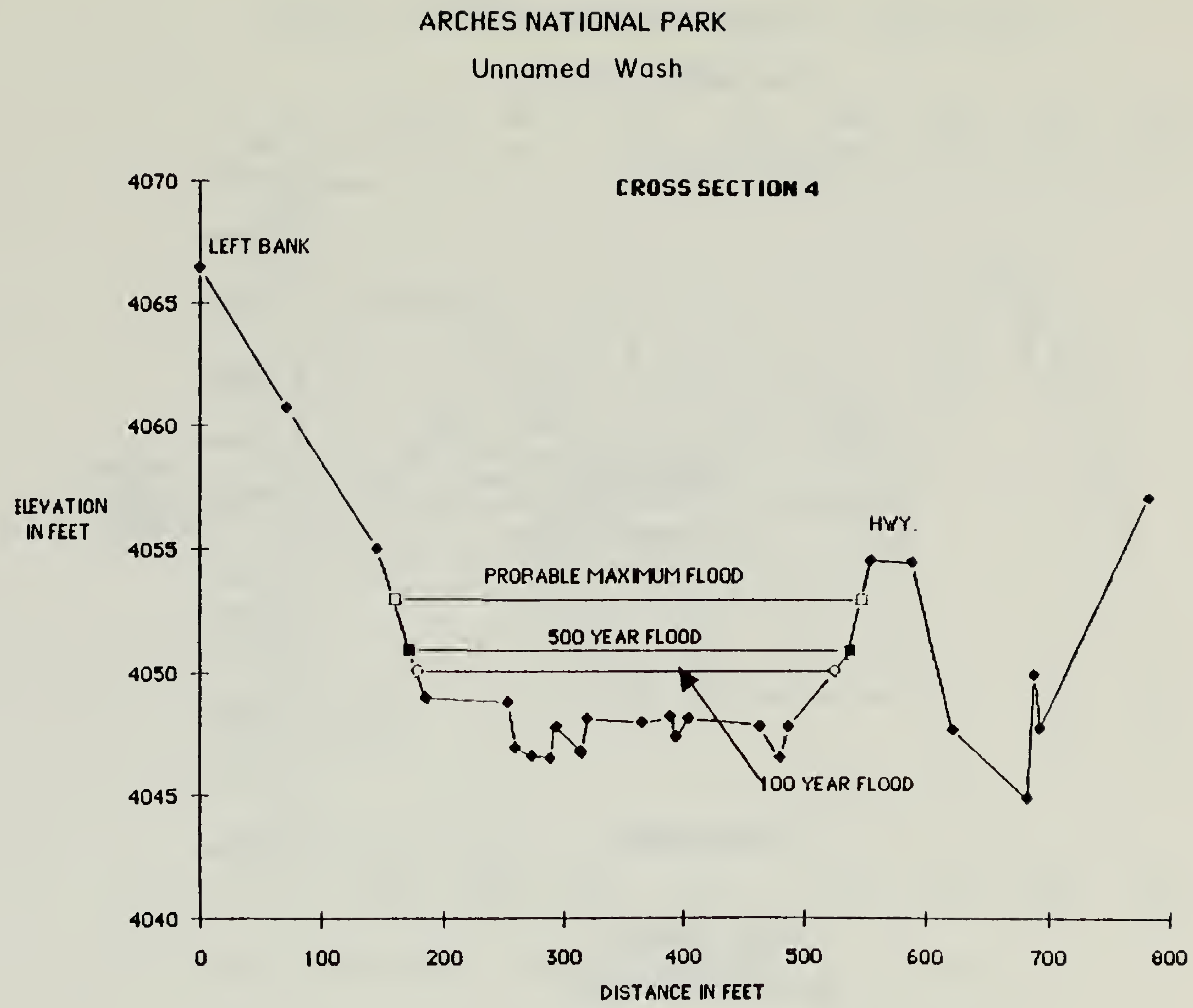

Figure 13. Floodwater - Surface Elevations -- Cross Section 4

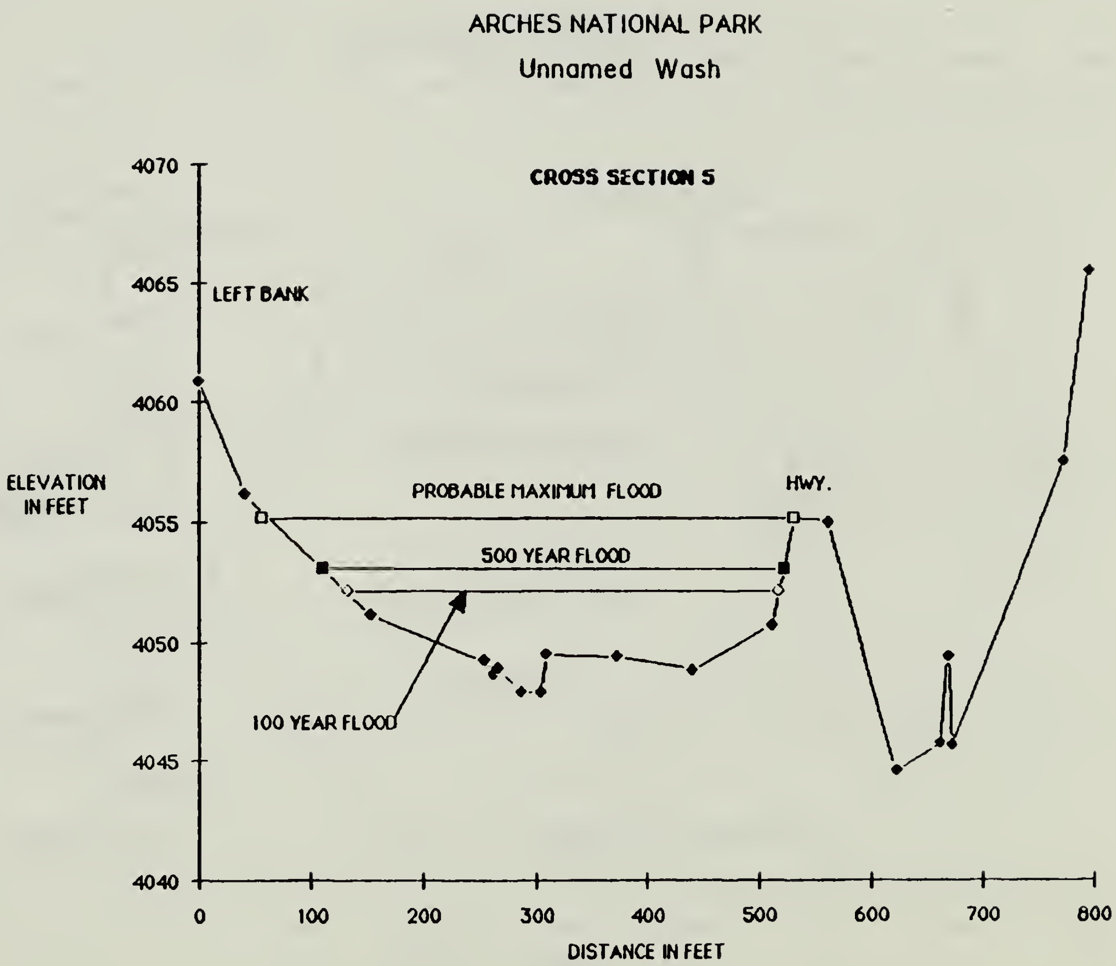

Figure 14. Floodwater - Surface Elevations -- Cross Section 5

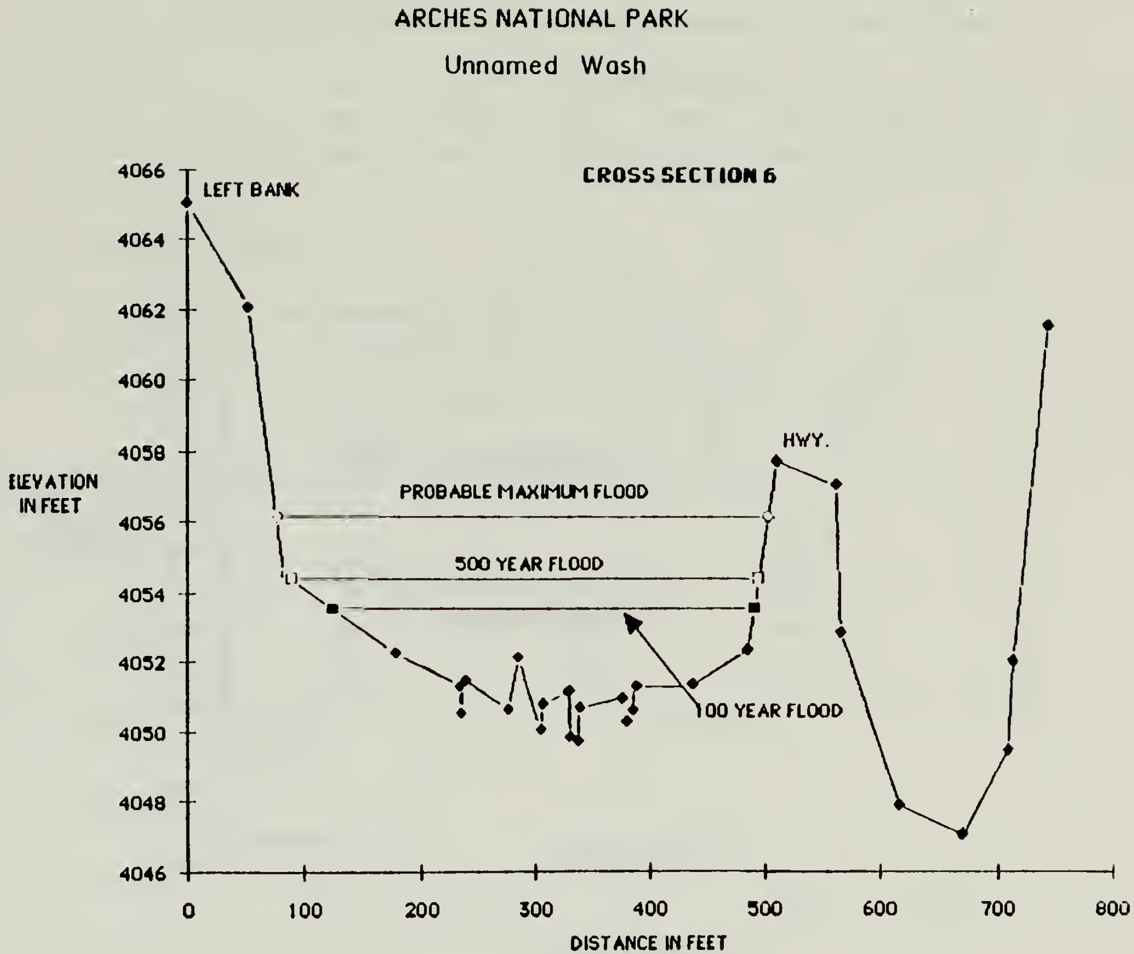

Figure 15. Floodwater - Surface Elevations -- Cross Section 6

ARCHES NATIONAL PARK

Unnamed Wash

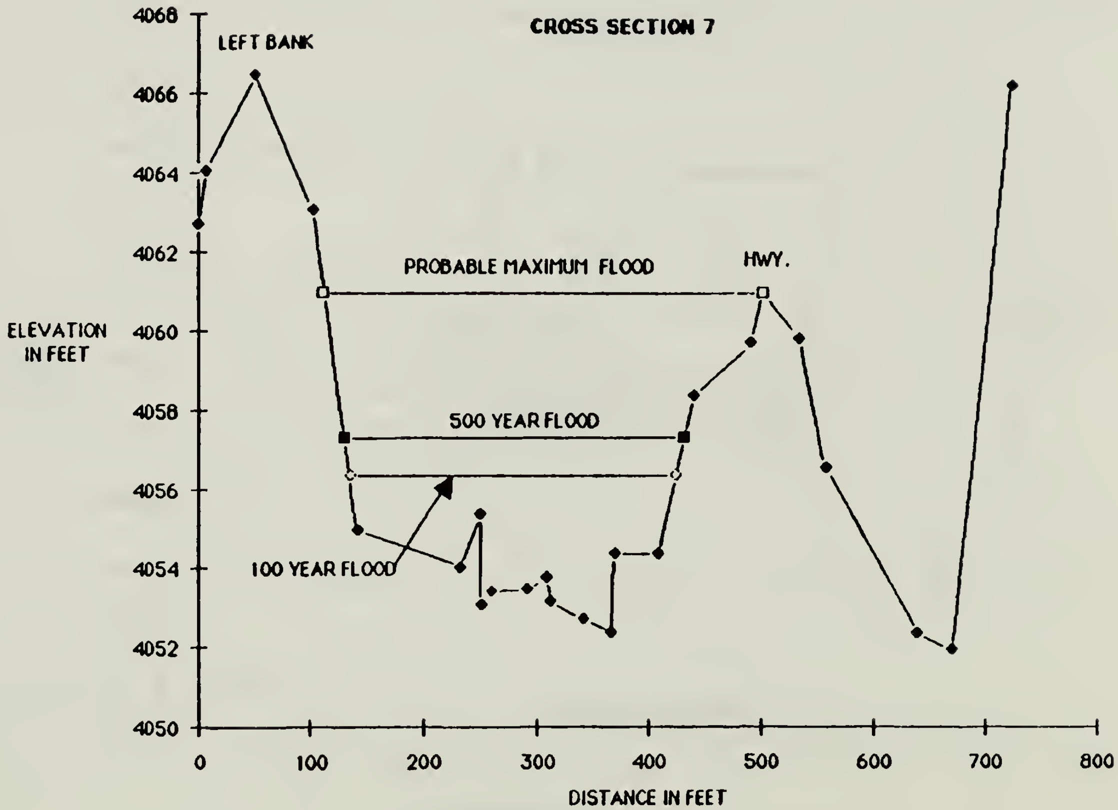

Figure 16. Floodwater - Surface Elevations -- Cross Section 7

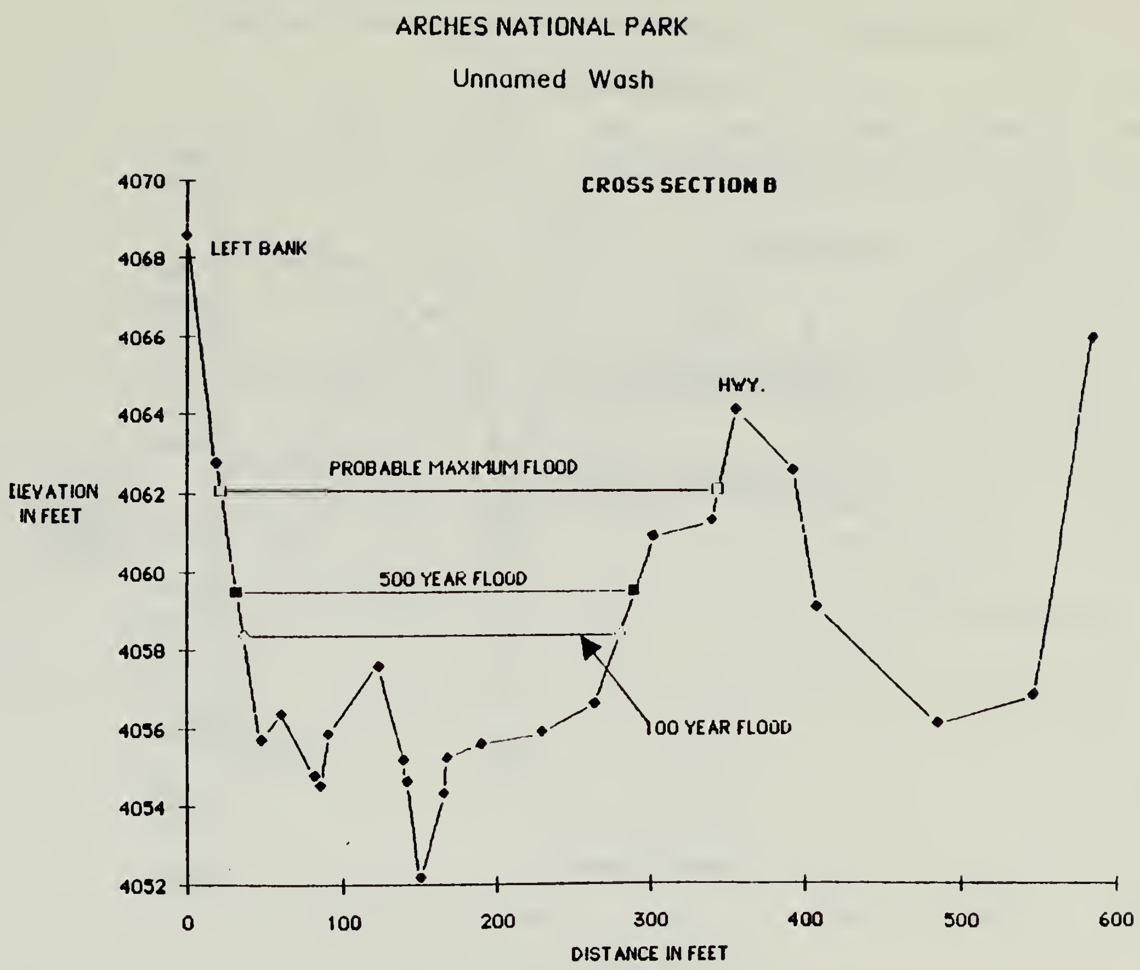

Figure 17. Floodwater - Surface Elevations -- Cross Section 8

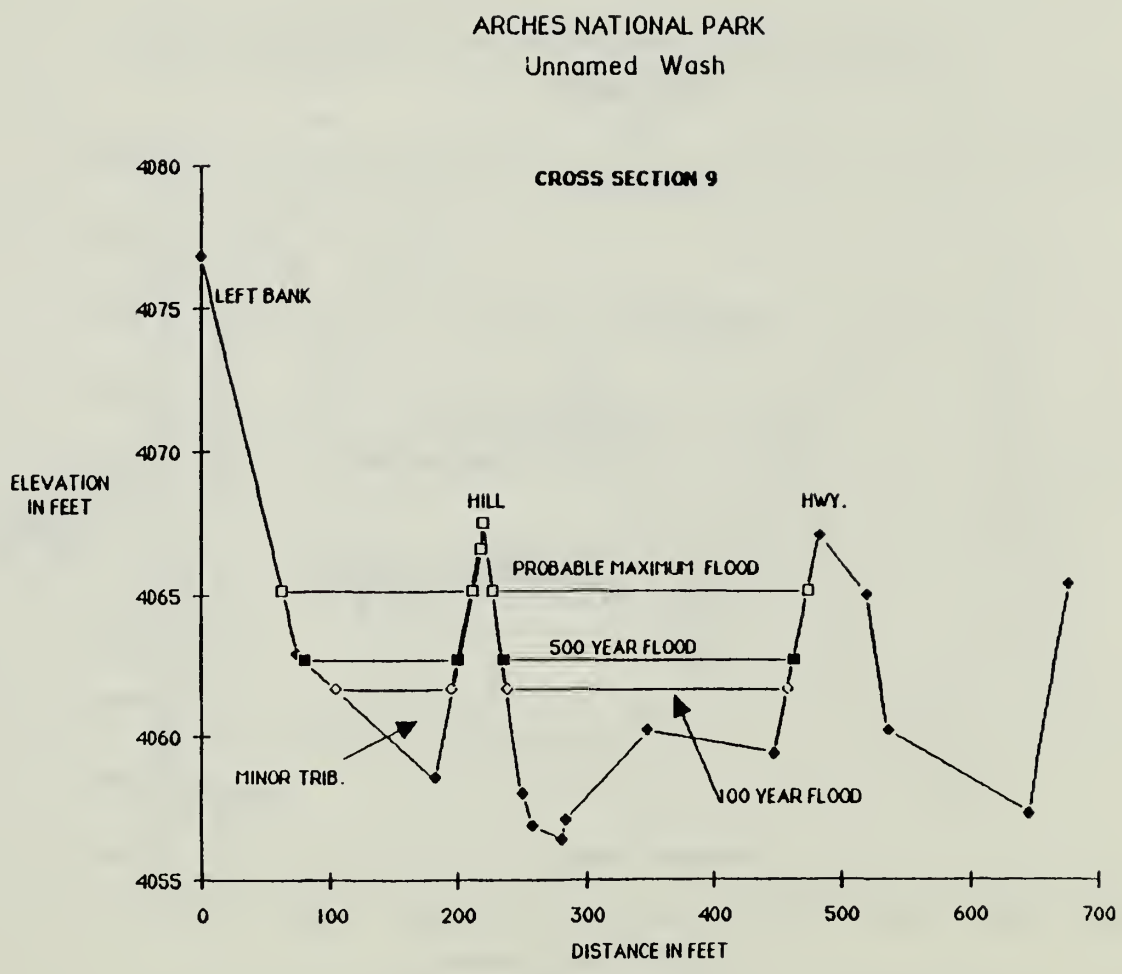

Figure 18. Floodwater - Surface Elevations -- Cross Section 9

ARCHES NATIONAL PARK

Unnamed Wash

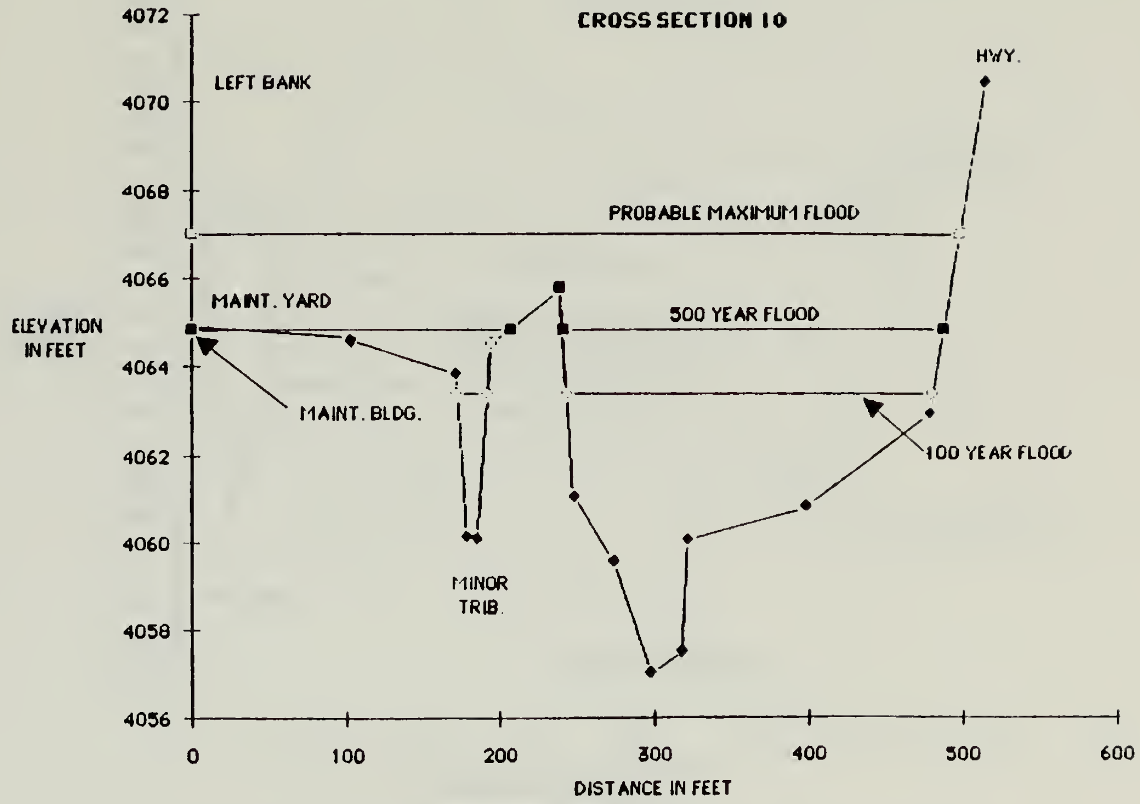

Figure 19. Floodwater - Surface Elevations -- Cross Section 10

ARCHES NATIONAL PARK
Unnamed Wash

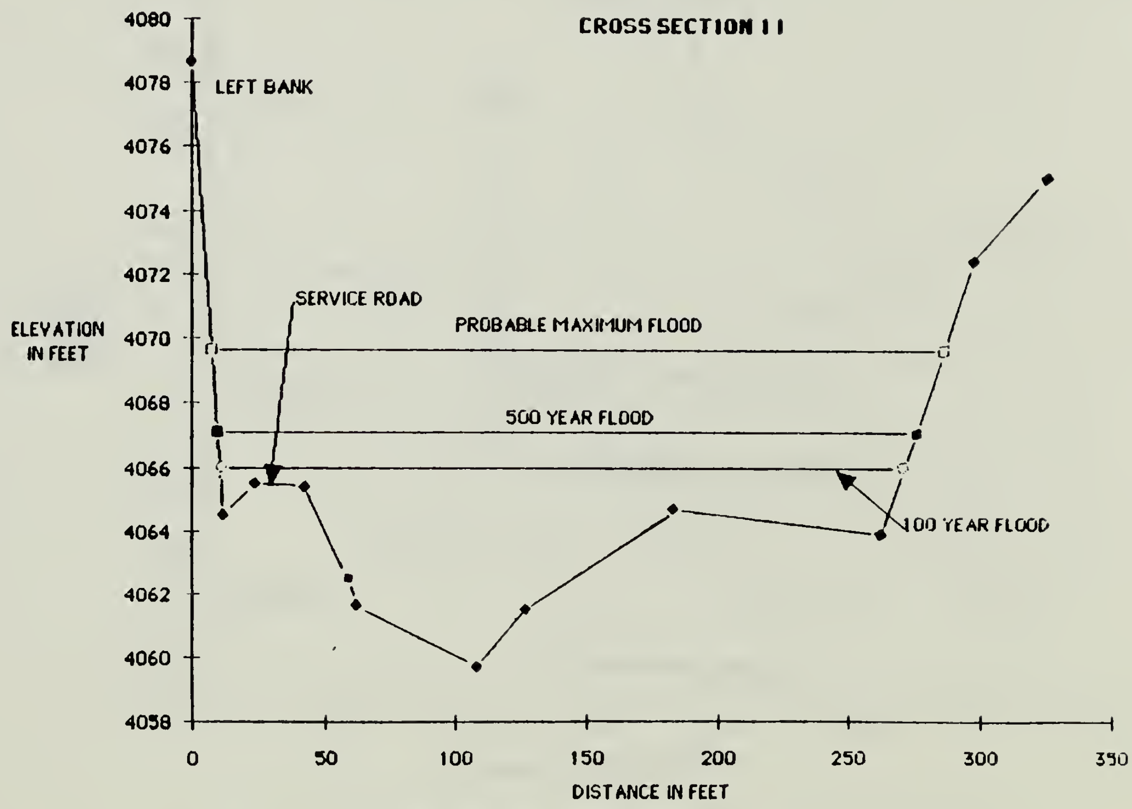

Figure 20. Floodwater - Surface Elevations -- Cross Section II

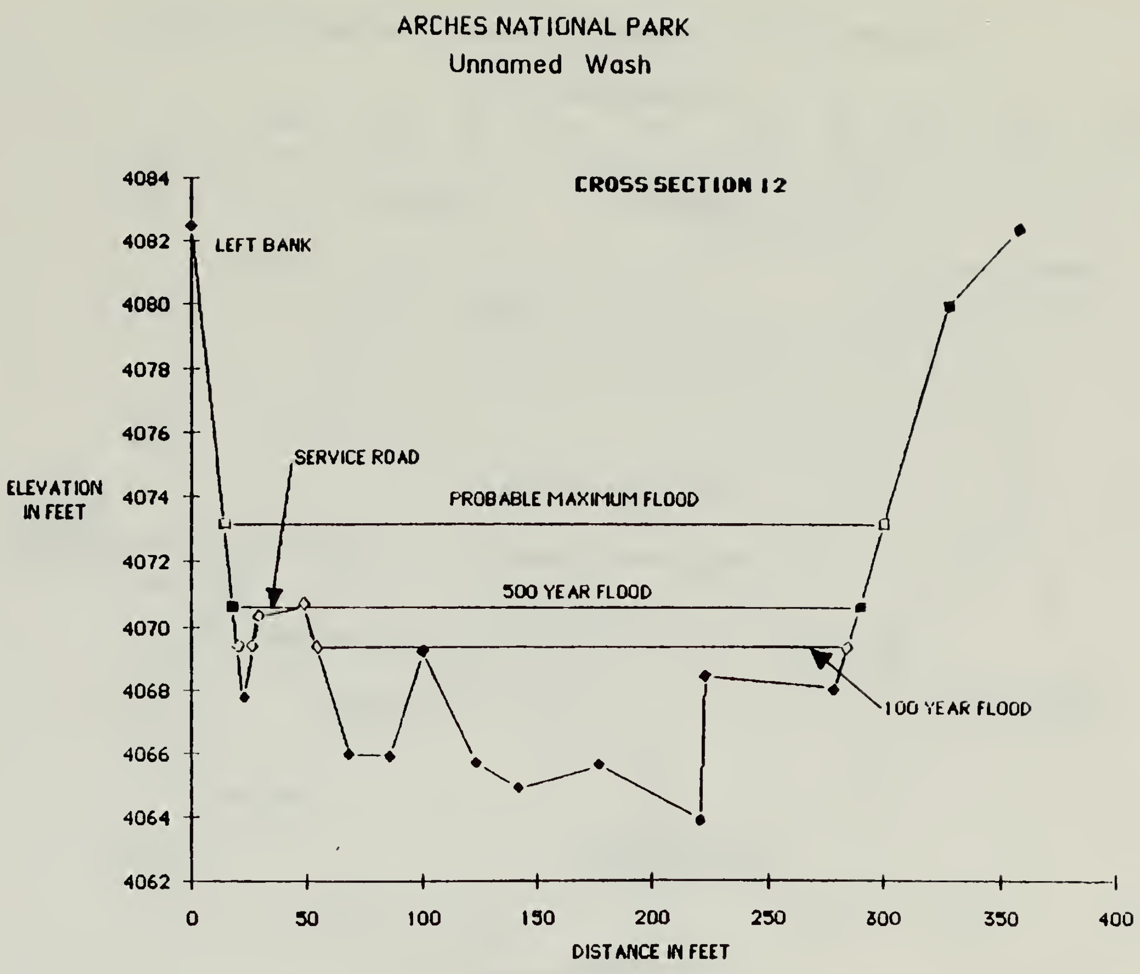

Figure 21. Floodwater - Surface Elevations -- Cross Section 12

ARCHES NATIONAL PARK
Unnamed Wash

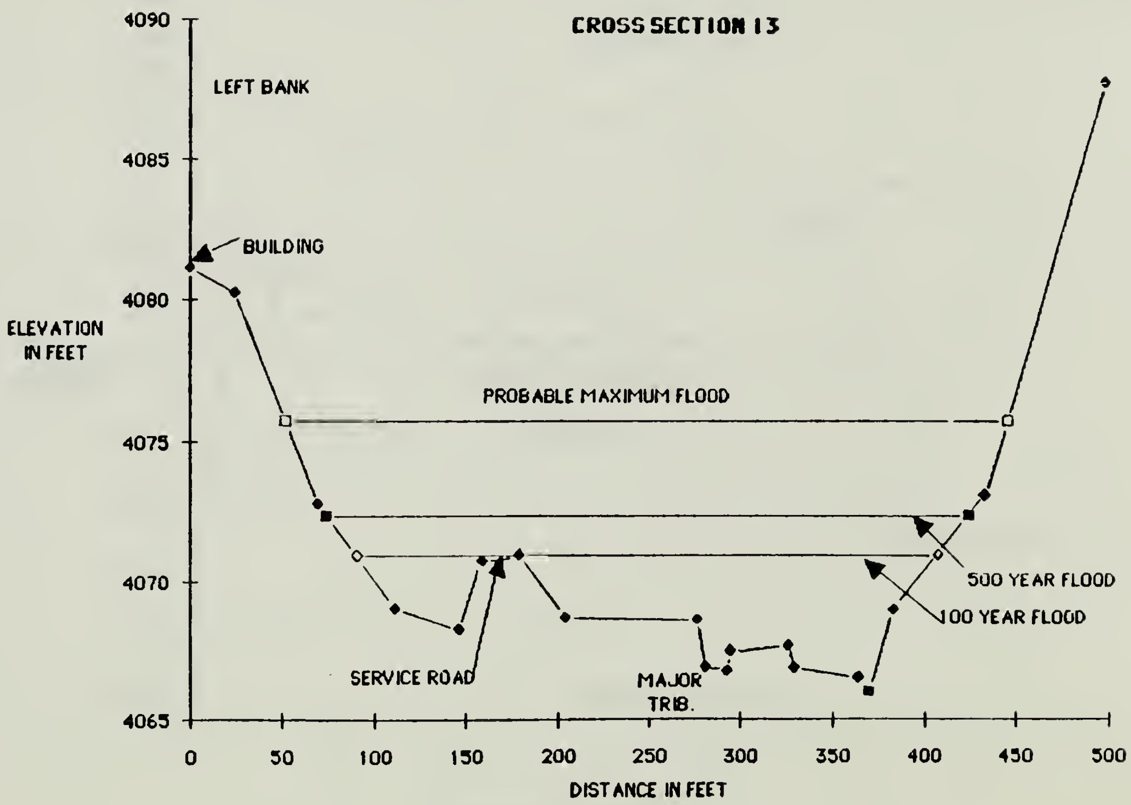

Figure 22. Floodwater - Surface Elevations -- Cross Section 13

ARCHES NATIONAL PARK

Unnamed Wash

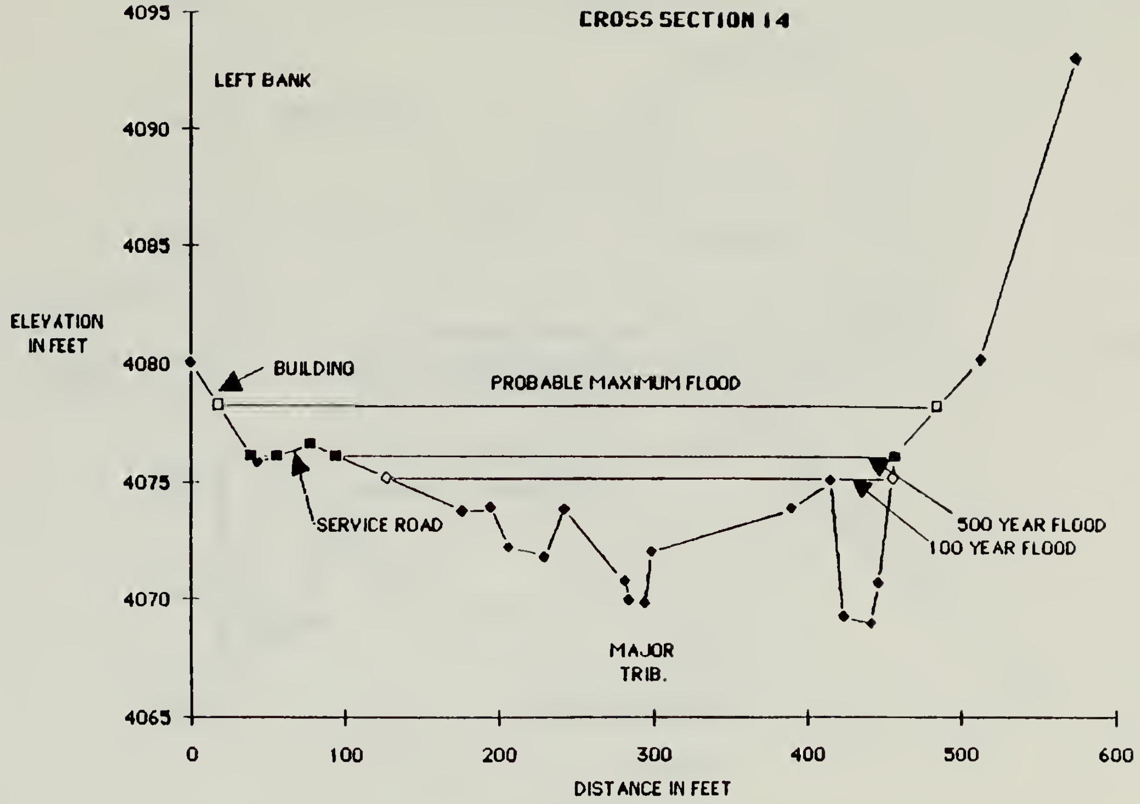

Figure 23. Floodwater -- Surface Elevations -- Cross Section 14

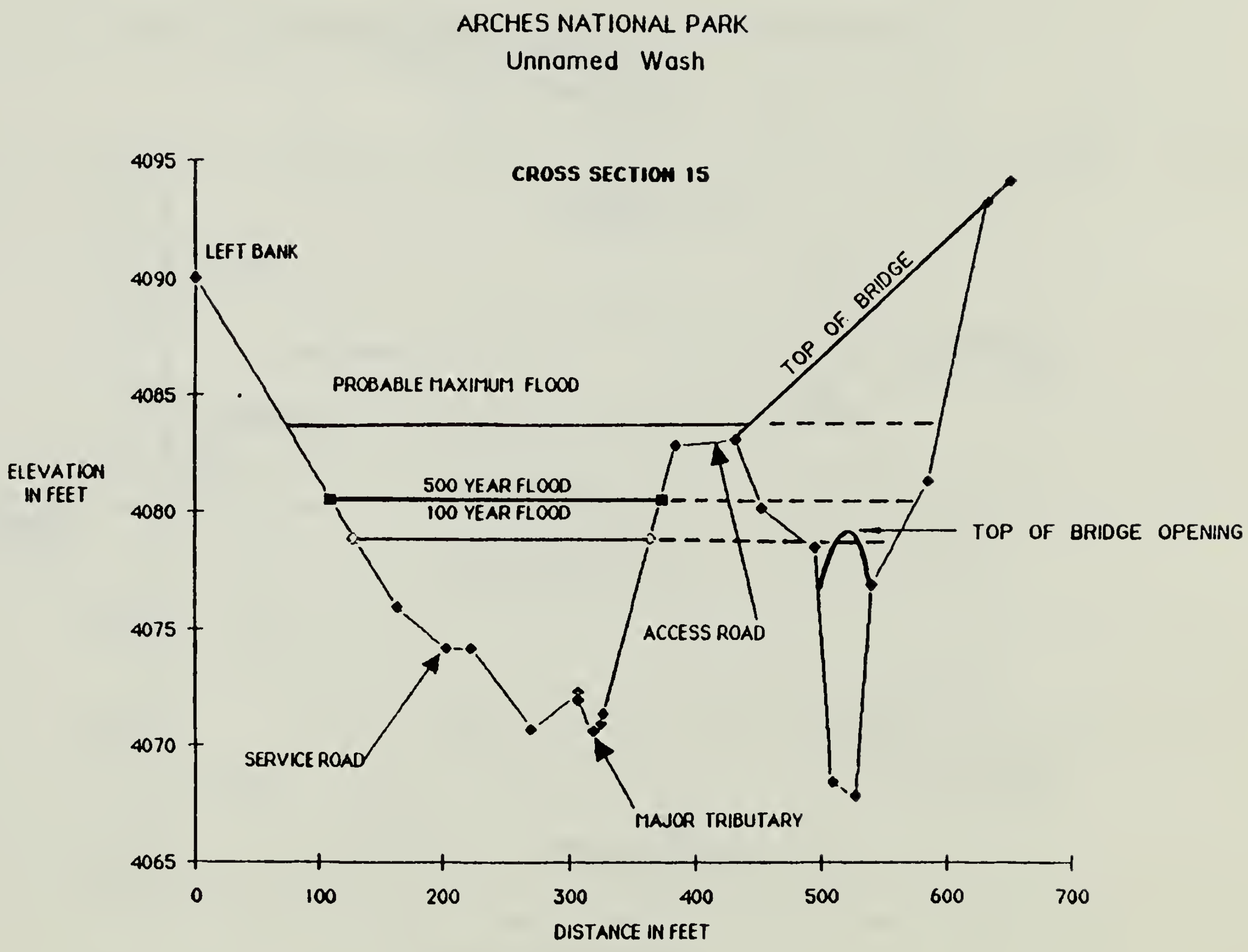

Figure 24. Floodwater - Surface Elevations -- Cross Section 15

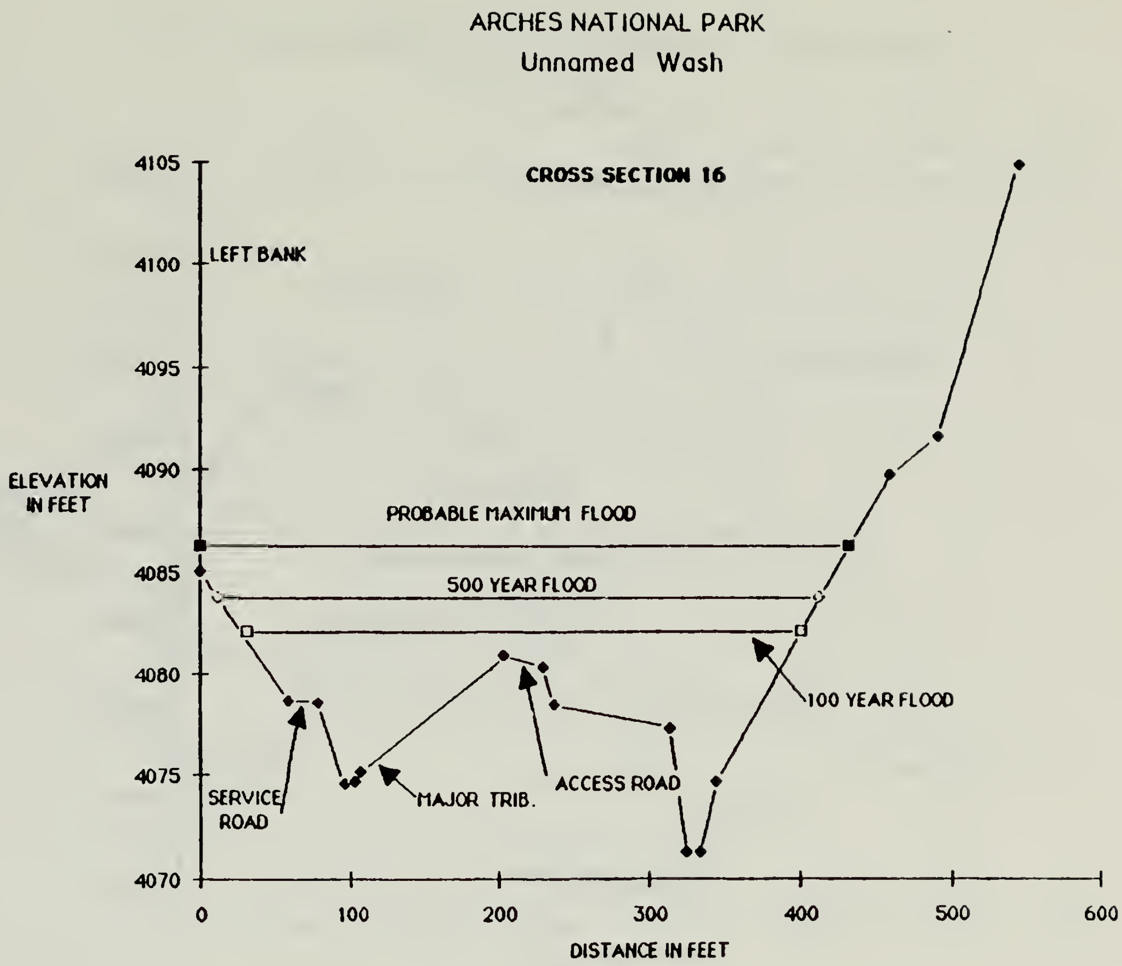

Figure 25. Floodwater - Surface Elevations -- Cross Section 16

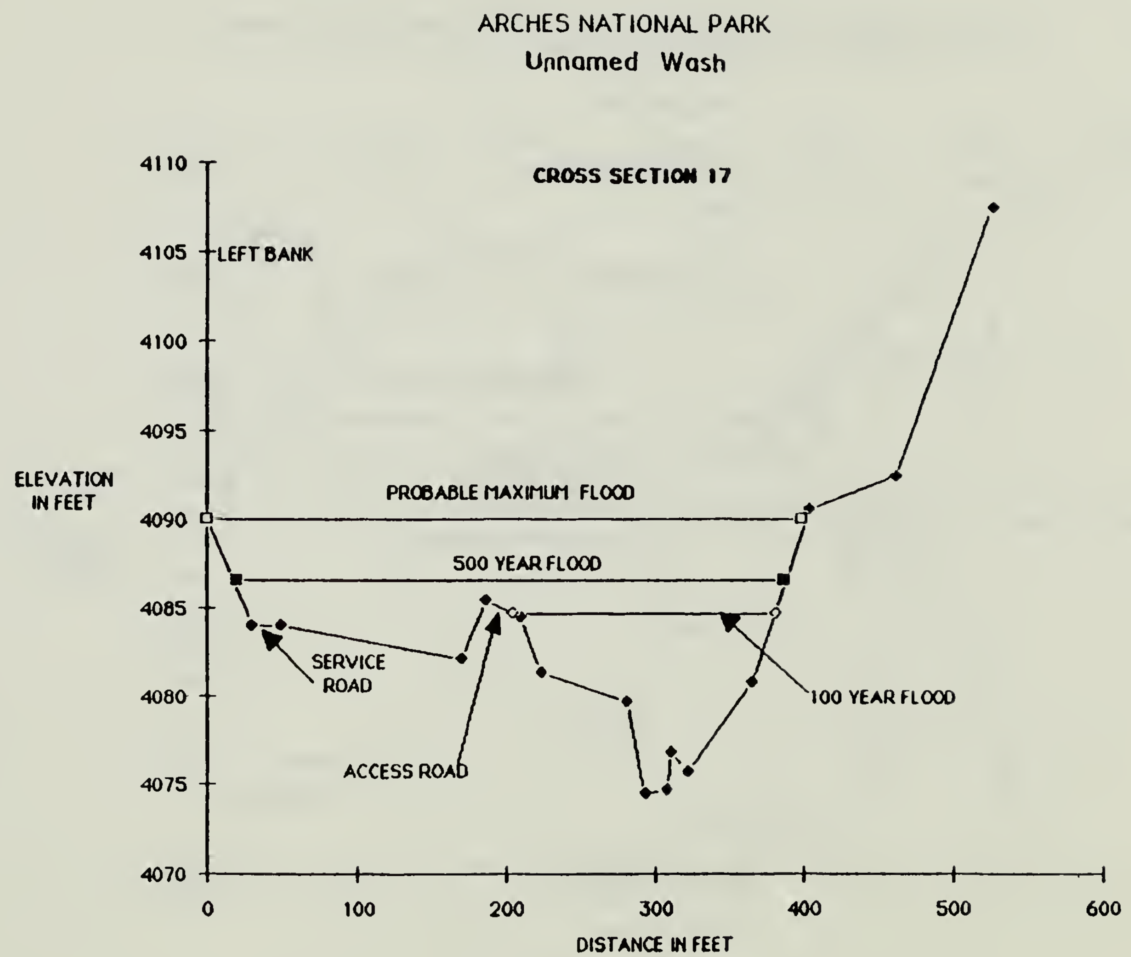

Figure 26. Floodwater - Surface Elevations -- Cross Section 17

ARCHES NATIONAL PARK

Unnamed Wash

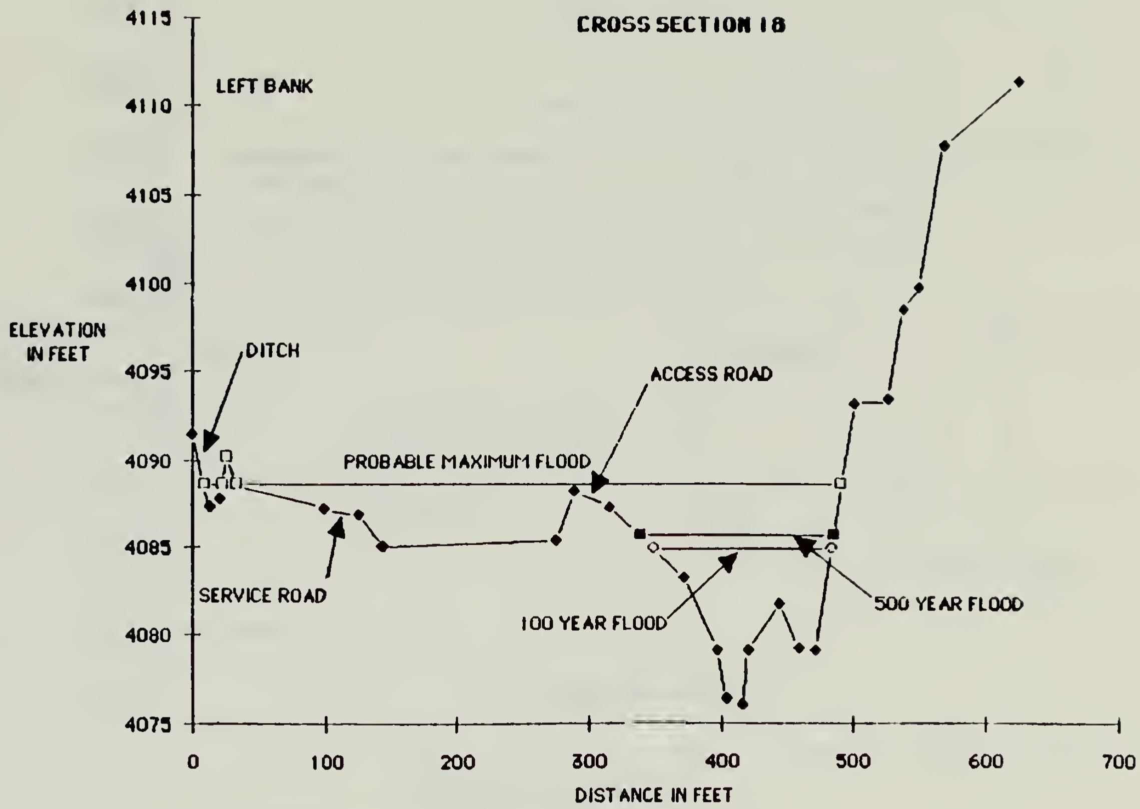

Figure 27. Floodwater - Surface Elevations -- Cross Section 18

ARCHES NATIONAL PARK

Unnamed Wash

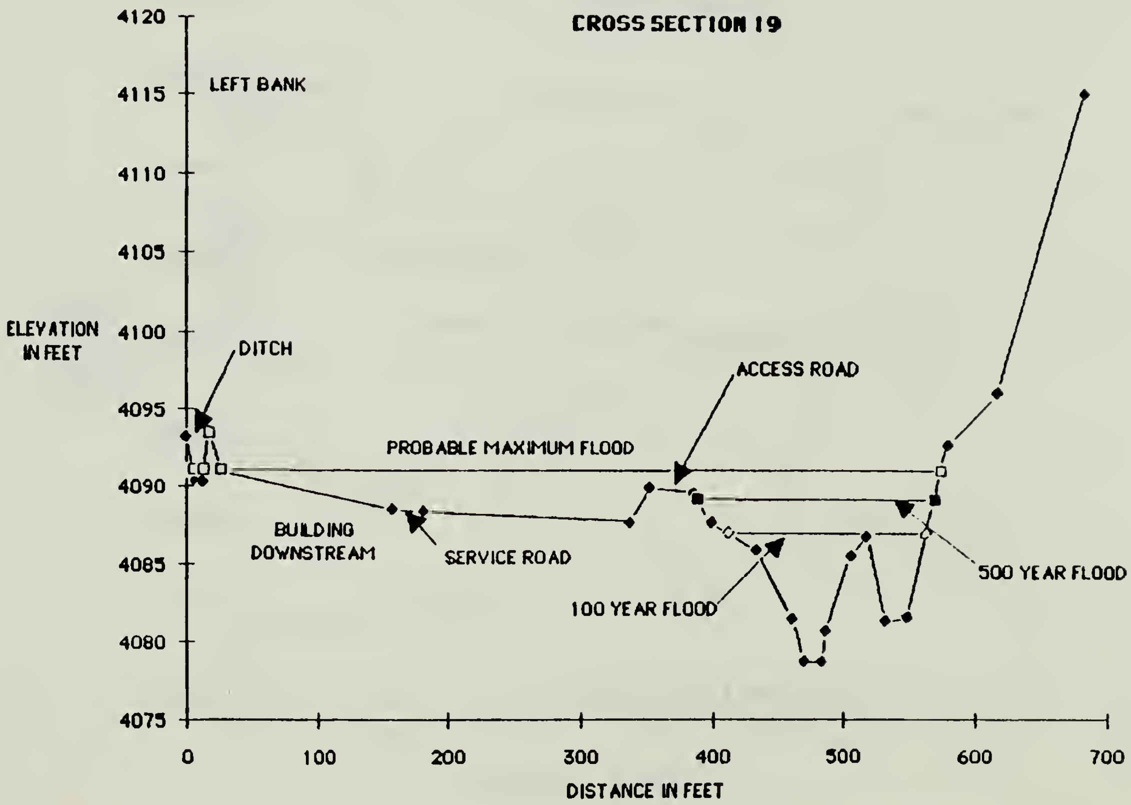

Figure 28 Floodwater - Surface Elevations -- Cross Section 19

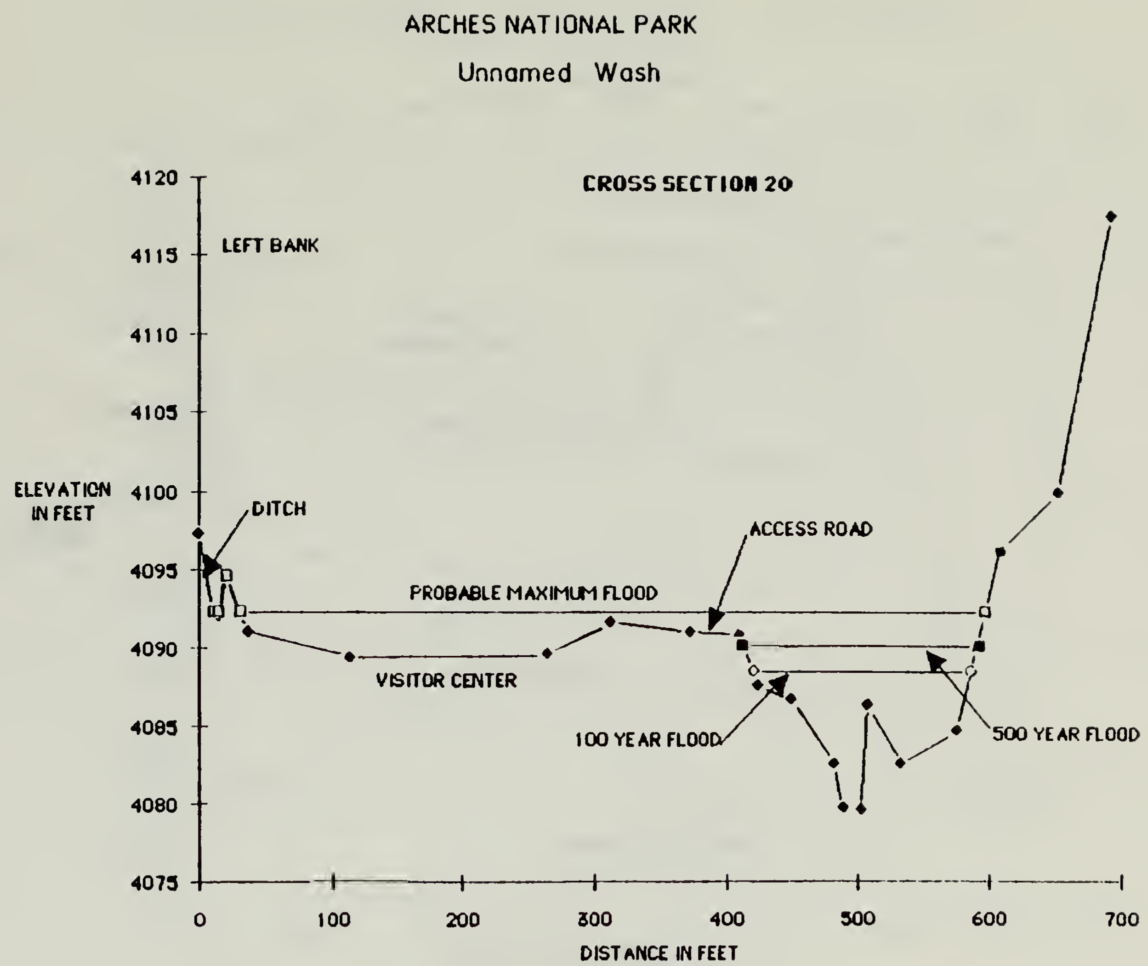

Figure 29. Floodwater - Surface Elevations -- Cross Section 20

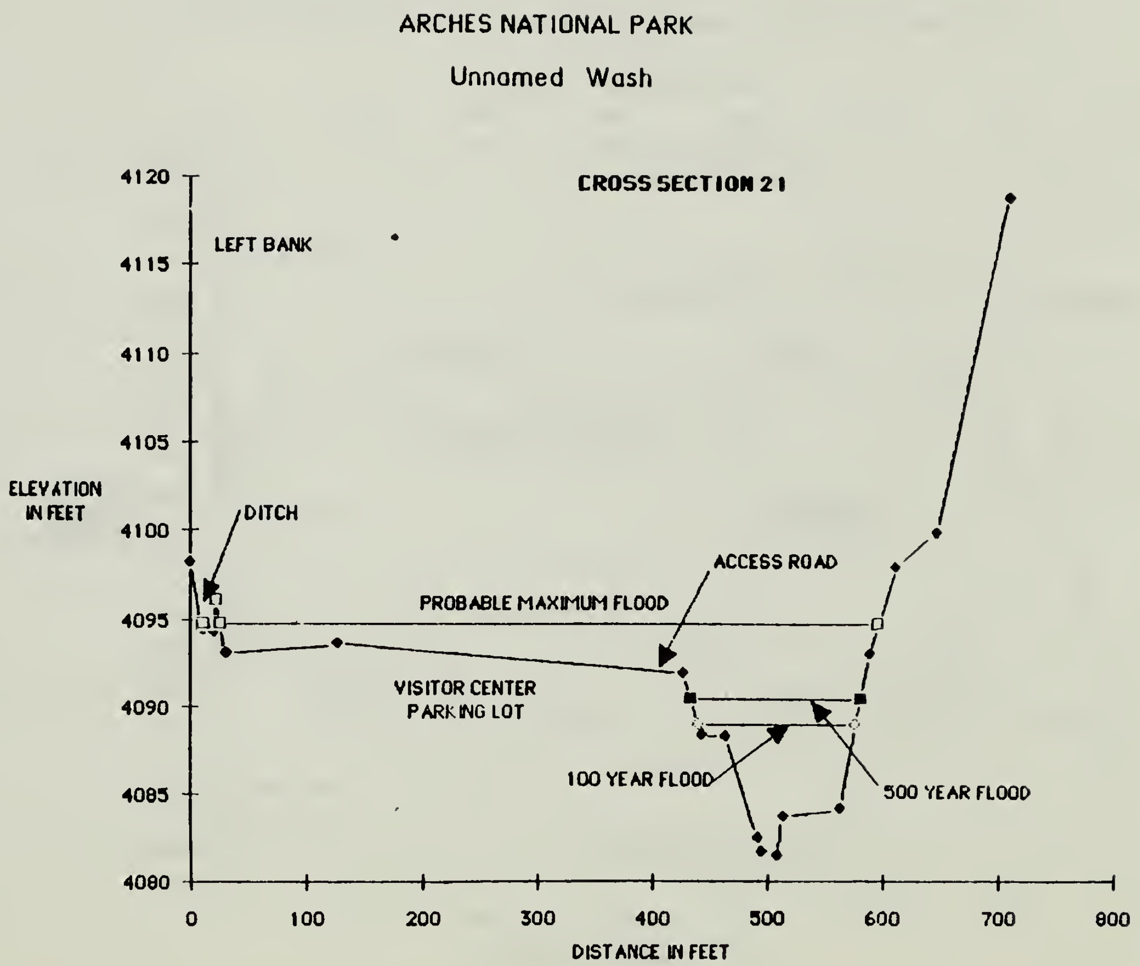

Figure 30. Floodwater - Surface Elevations -- Cross Section 21

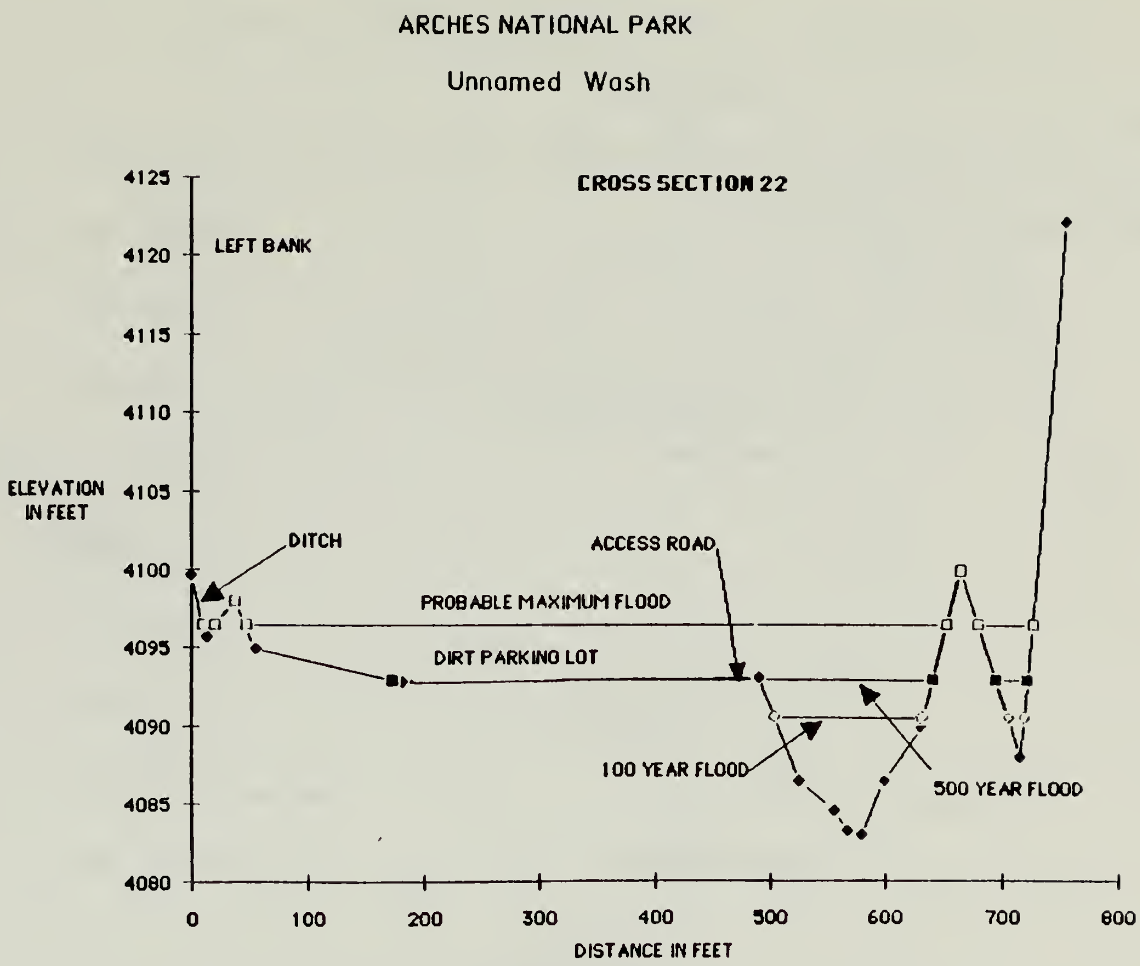

Figure 31. Floodwater - Surface Elevations -- Cross Section 22

ARCHES NATIONAL PARK
Unnamed Wash

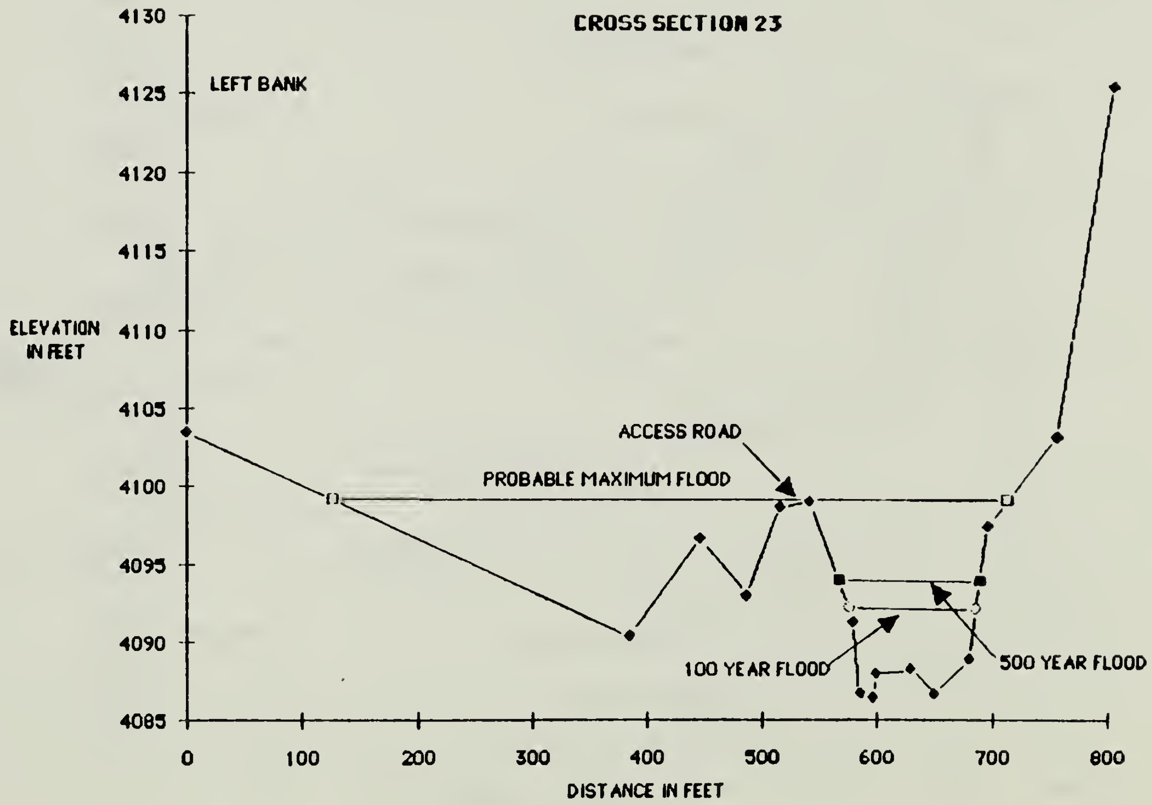

Figure 32. Floodwater - Surface Elevations -- Cross Section 23

ARCHES NATIONAL PARK
Unnamed Wash

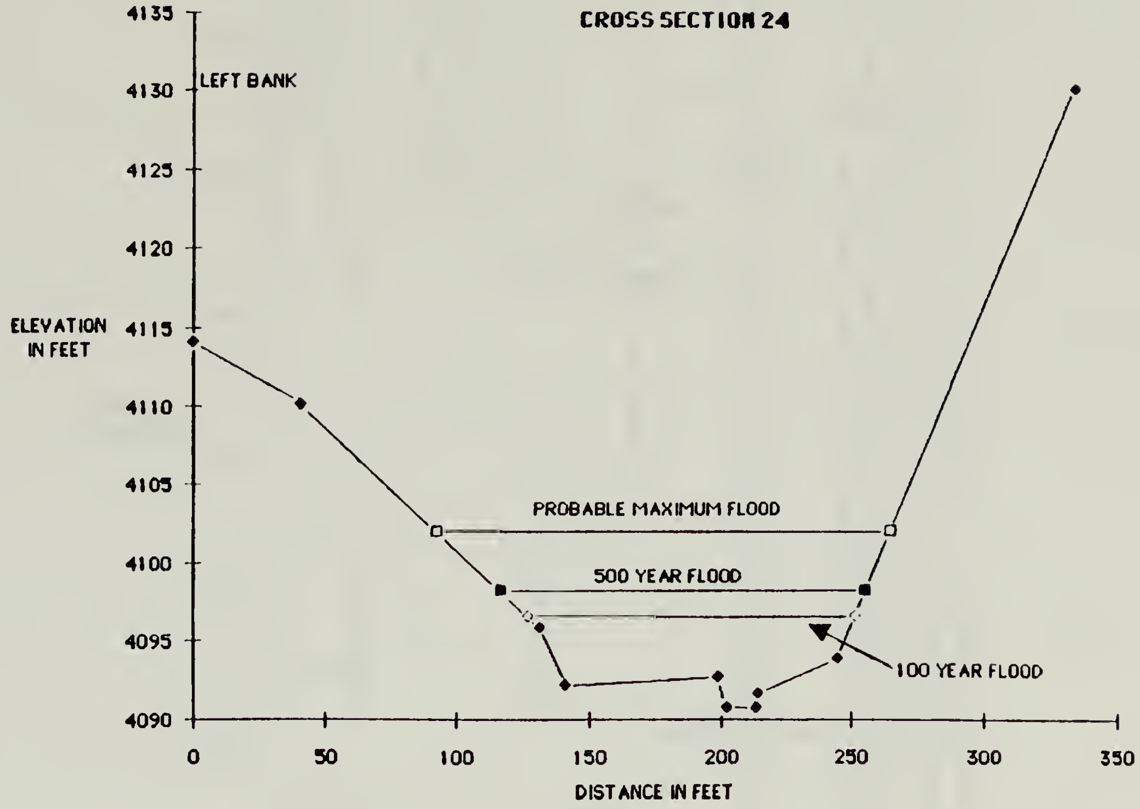

Figure 33. Floodwater - Surface Elevations -- Cross Section 24

ARCHES NATIONAL PARK
Unnamed Wash

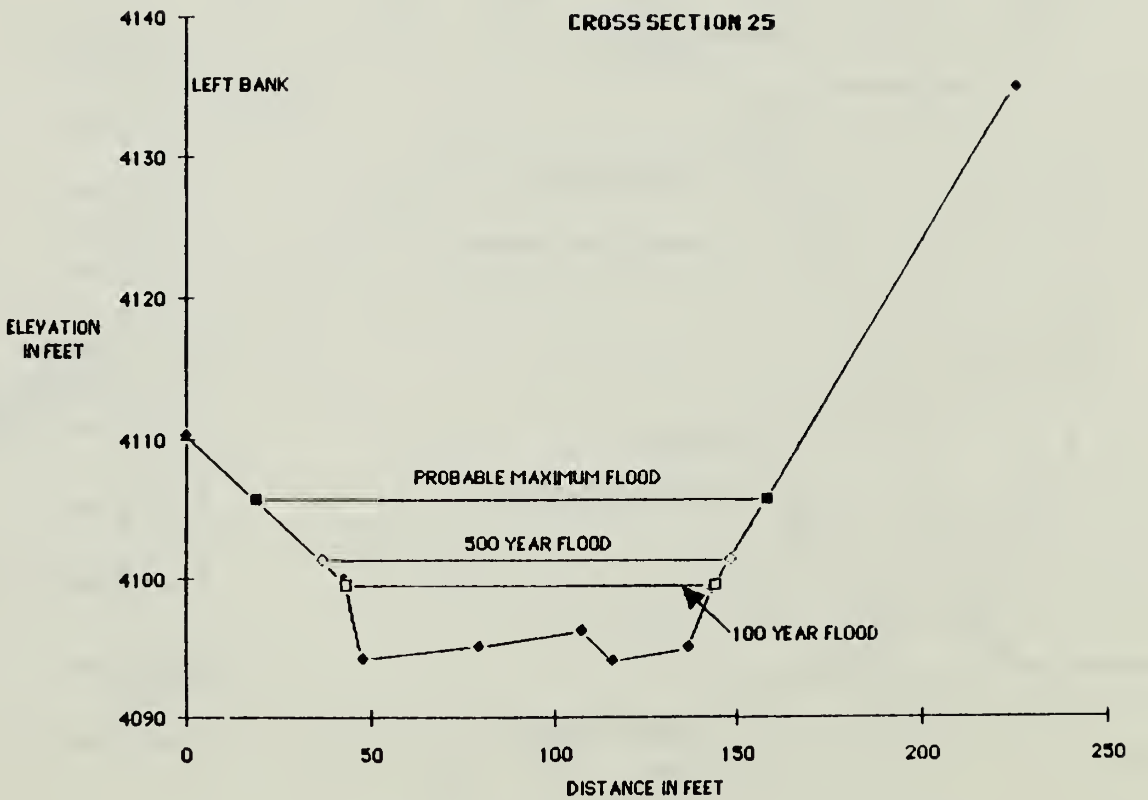

Figure 34. Floodwater - Surface Elevations -- Cross Section 25

The National Park Service Water Resources Division is responsible for providing water resources management policy and guidelines, planning, technical assistance, applied research, training and operational support to units of the National Park Service. Program areas include water rights, water resources planning, regulatory guidance and review, hydrology, water quality, watershed management, watershed studies and aquatic ecology.

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Copies of this report are available from the following:

Computer Assistant (303) 221-8330
National Park Service
Water Resources Division
301 S. Howes Street
Fort Collins, CO 80521

Technical Information Center (303) 969-2130
Denver Service Center
P.O. Box 25287
Denver, CO 80225-0287

As the nation's principal conservation agency, the Department of the Interior has the responsibility for most of our nationally owned public lands and natural and cultural resources. This includes fostering wise use of our land and water resources, protecting our fish and wildlife, preserving the environmental and cultural values of our national parks and historical places, and providing for enjoyment of life through outdoor recreation. The department assesses our energy and mineral resources and works to ensure that their development is in the best interests of all our people. The department also promotes the goals of the Take Pride in America campaign by encouraging stewardship and citizen responsibility for the public lands and promoting citizen participation in their care. The department also has a major responsibility for American Indian reservation communities and for people who live in island territories under U.S. administration.

The mission of the Water Resources Division is to preserve and protect National Park Service water resources and water dependent environments. This mission is accomplished through a watershed management program based on needs at the park, Region, and National levels.

NPS D-45A October 1990